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WATER & WASTEWATER ENGINEERING DIVISION

Outfall Collection System Evaluation and Sewershed Plan Project 1039

Alternative Analysis and Recommendations Sanitary Sewer Overflow Consent Decree

Civil Action No. JFM-02-1524

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EXECUTIVE SUMMARY

The Alternative Analysis and Recommendation Report (AARR) for the Outfall Sewershed is a discussion of the development and evaluation of facilities for three alternatives that eliminate SSOs in the Outfall Sewershed. The objectives of the Consent Decree relevant to the AARR are defined in the BaSES Manual, particularly sections 7.7, 7.8.3, and 8.2. The alternatives developed in this report define improvements that mitigate SSOs for design storms of increasing severity. The model configuration represents future conditions (year 2025) and planned improvements.

The Outfall Sewershed is unique among all of the Baltimore sewersheds in that most of the flows conveyed through the Outfall Sewershed network originate from upstream sewersheds (Jones Falls, High Level, Low Level, Herring Run, and Dundalk). A relatively small fraction of the flow originates from the subcatchment areas within the Outfall Sewershed. Consequently, the largest and most costly alternative facilities are sized to accommodate the high flows from upstream sewersheds. Conveyance improvements in the upstream sewersheds have the potential to increase the risk of SSOs in the Outfall Sewershed and have a direct influence on the size and cost of the required alternative facilities.

The Baseline Analysis and Capacity Assessment (BACA) identified locations of overflows in the Outfall Sewershed and discussed causes of those SSOs. The report presented the risk of overflows for Future 2025 conditions; however, at the time of writing the BACA report, the boundary conditions applied to the Outfall Sewershed model for the Future 2025 conditions did not reflect recommended facilities in upstream sewersheds that had the potential to increase flows to the Outfall Sewershed.

Upstream improvements result in more severe flows and greater SSO volumes than those presented in the BACA report. The Future 2025 results in the BACA report are useful in that they identify locations with a SSO risk and sections of pipes that have hydraulic restrictions. The qualitative results are informative, but the numerical magnitude of overflow volumes and peak overflow rates in the BACA report for the Future 2025 condition are based on the original boundary conditions which produce significantly smaller simulated overflows. Consequently, this AARR is based on a revised analysis using the “Upstream Improvements” boundary conditions.

The largest overflows in the Outfall Sewershed are located on or near the 99-inch sewer and are caused by conveyance limitations of the large diameter trunk sewers and high inflows from upstream sewersheds. Smaller overflows occur along the smaller branch sewers due to localized hydraulic restrictions and high flows generated in the subcatchment areas.

The AARR briefly discusses alternatives to resolve SSOs in the smaller branch sewers of meter basins HL02, HL04, HL05, and OUT01. Most of these branch sewer SSOs are caused by local hydraulic restrictions and are resolved by increasing the pipe size.

Most of the attention in the AARR is given to alternatives that resolve SSOs along the trunk sewers. Relief is needed at the upstream end of the Outfall Interceptor and at the upstream end of the 99-inch sewer. Peak wet weather discharge rates from the Eastern Avenue Pump Station exceed the conveyance capacity of the 99-inch sewer. The excess flow is primarily relieved by an overflow at Bethel and Moyer Streets. All of the alternatives in the AARR propose an overflow weir at the upstream end of the 99-inch sewer in the vicinity of Fayette and Bond Streets. Relief is also needed at the upstream end of the Outfall Interceptor in the vicinity of Chase and Durham Streets. Peak flows from the High Level Sewershed, along with flow from the 99-inch sewer exceed the conveyance capacity of the Outfall Interceptor. A relief facility at Chase and Durham Streets would protect against SSO in this vicinity (especially at a low manhole along Durham Street near Eager Street).

All of the alternatives assume that sediment is removed from the 99-inch sewer, Outfall Interceptor, and Outfall Relief sewer. Sediment removal increases the conveyance capacity by restoring the full cross section area to flow and reducing the hydraulic roughness of pipes.

Alternative 1 proposes two storage tanks, one at Fayette and Bond Streets and the other at Chase and Durham Streets, to attenuate the peak flows in the trunk sewers. Excess flows enter the storage tanks so that the remaining flows are within the conveyance capacities of the pipes without sediment. Unlike the other alternatives, Alternative 1 does not assume any changes downstream at the Back River WWTP. The existing treatment capacity limits flow and causes surcharged conditions in the Outfall Interceptor at the County Line.

Alternative 2 assumes that downstream improvements are in place. These would be improvements that increase the treatment capacity of the Back River WWTP. The downstream improvements are represented in the Outfall Sewershed model as a downstream level boundary condition at the County Line that does not exceed 48 feet. At 48 feet the Outfall Interceptor and Outfall Relief sewer are approximately 90% full with the water level one foot below the crown of the pipe. The downstream improvements greatly increase the conveyance capacity and reduce the volume of storage required. As a result no storage is needed for the 2-year event and only one storage tank is needed for the 5, 10, 15, and 20-year events. The tank is located at the Fayette relief site and is much smaller than the size of the tanks used in Alternative 1.

Alternative 3 also assumes that downstream improvements are in place. Alternative 3 uses a tunnel from the proposed Fayette Street relief facility to a proposed reconnection point along Lombard Street near to the connection from the Dundalk Sewershed.

The total estimated costs of the three alternative options are compared for the 10-year return period design storm. Alternative 2 with a small storage tank is the lowest cost option. Alternative 3 with a tunnel is more expensive but it provides greater protection against SSOs and greater flexibility for future operations and maintenance. No specific recommendations are made in the report at this time. The material in this report is to be used to facilitate ongoing discussion of the alternative concepts with the City and the Technical Program Manager.

In support of those discussions, itemized costs are developed for Alternative 3 facilities for all of the design storm levels of protection. The total estimated costs for the Outfall Sewershed improvements are summarized in the tables below for the 2, 5, 10, 15, and 20-year events. Costs are inflated 7% per year for the recommended projects depending on the year they could be implemented (from 2008 through 2017). The tables also contain the additional cost to improve SSO control from one design storm level of protection to the next.

This report also contains results of a sensitivity study that examines the risk of failing to achieve the desired level of protection against overflows due to variations in key modeling parameters. In particular, the sensitivity study evaluated the storage required depending on the assumed value of hydraulic roughness (Manning's n value) of the pipes after they are cleaned of sediment and the response to operations of the Eastern Avenue Pump Station. A more robust alternative configuration is identified that is likely to perform well for sub-optimal parameter values and during extreme wet weather conditions. The 2-year level of protection can still be achieved under sub-optimal conditions if the size of the alternative facilities is equal to the size identified for the 10-year event under more desirable modeling assumptions.

Total Estimated Outfall Improvement Costs										
Projected Year	2-yr Cost	5-yr		10-yr		15-yr		20-yr		
		Additional	Cumulative	Additional	Cumulative	Additional	Cumulative	Additional	Cumulative
2008	$24,282,000	$110,629,000	$134,911,000	$14,595,000	$149,506,000	$15,085,000	$164,591,000	$880,000	$165,471,000
2009	$25,982,000	$118,373,000	$144,355,000	$15,616,000	$159,971,000	$16,141,000	$176,112,000	$942,000	$177,054,000
2010	$27,801,000	$126,659,000	$154,460,000	$16,709,000	$171,169,000	$17,271,000	$188,440,000	$1,008,000	$189,448,000
2011	$29,747,000	$135,525,000	$165,272,000	$17,879,000	$183,151,000	$18,480,000	$201,631,000	$1,078,000	$202,709,000
2012	$31,829,000	$145,012,000	$176,841,000	$19,131,000	$195,972,000	$19,773,000	$215,745,000	$1,154,000	$216,899,000
2013	$34,057,000	$155,163,000	$189,220,000	$20,470,000	$209,690,000	$21,157,000	$230,847,000	$1,235,000	$232,082,000
2014	$36,441,000	$166,024,000	$202,465,000	$21,903,000	$224,368,000	$22,638,000	$247,006,000	$1,322,000	$248,328,000
2015	$38,992,000	$177,646,000	$216,638,000	$23,436,000	$240,074,000	$24,222,000	$264,296,000	$1,415,000	$265,711,000
2016	$41,721,000	$190,082,000	$231,803,000	$25,076,000	$256,879,000	$25,918,000	$282,797,000	$1,514,000	$284,311,000
2017	$44,641,000	$203,388,000	$248,029,000	$26,832,000	$274,861,000	$27,732,000	$302,593,000	$1,620,000	$304,213,000

1.0 Project Description

1.1 Project location

The Outfall Sewershed is located in the east-central portion of the City of Baltimore, and encompasses approximately 3.62 square miles within the city boundaries. It is tributary to the Back River Wastewater Treatment Plant (WWTP) and its boundaries are approximately at the City-County line on the east, McElderry Avenue on the south, Bond Street on the west and Erdman Avenue on the north.

The Outfall Sewer system includes the approximately 336,040 linear feet (LF) of gravity sewers ranging from 8- to 144-inches in diameter; 1,845 manholes and structures; and 1 siphon.

1.2 Sub-sewersheds

Outfall Sewershed consists of a total of 16 sub-sewersheds as shown in Figure 1.1. The sub-sewersheds are named after the flow meter basins that correspond to the sub-sewersheds.

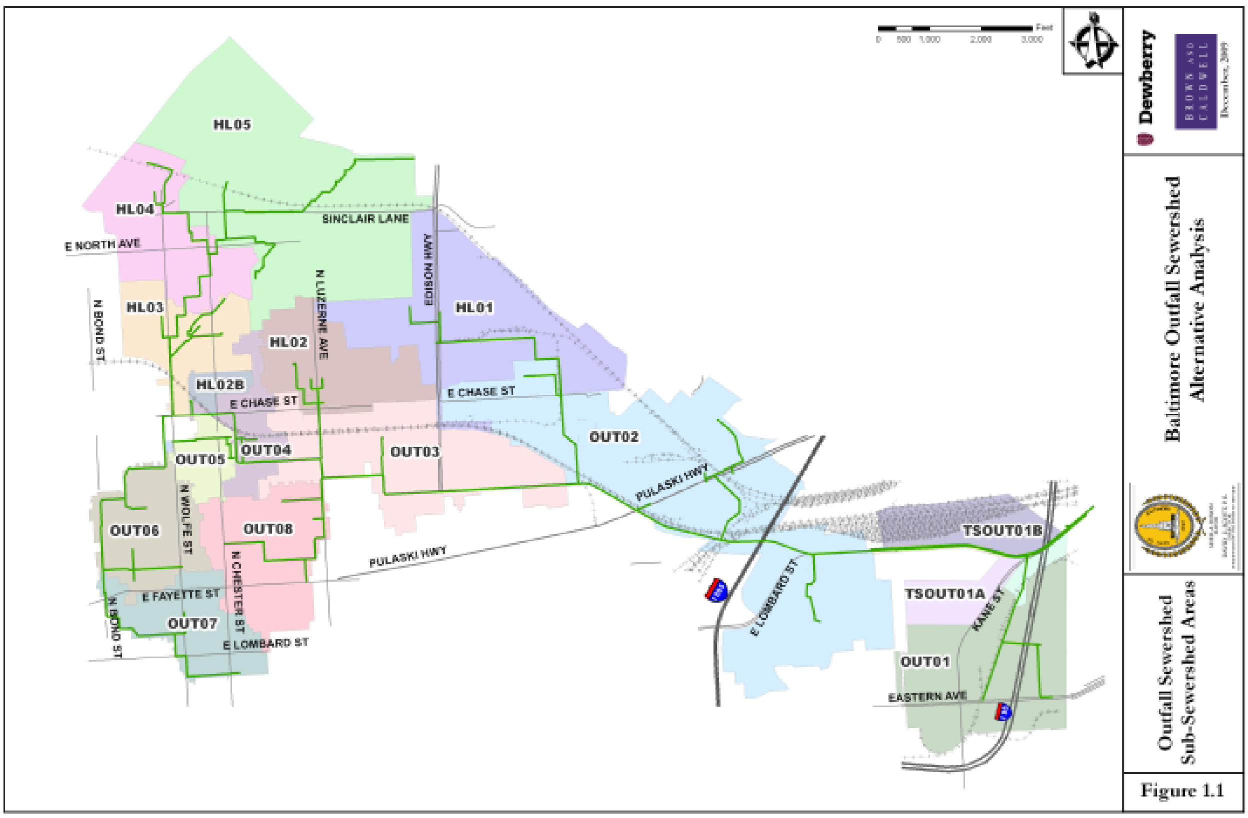


Figure 1.1

Figure 1.2 is a schematic of the large diameter trunk sewers in the Outfall Sewershed. The schematic shows the points of inflow to the Outfall Sewershed model from the upstream sewersheds and the locations of the downstream boundary conditions at the County Line. Approximate capacities of the trunk sewers are noted on the schematic for a clean condition if sediment were removed and for the existing condition with sediment. The capacities are given as ranges to account for the variable depth of sedimentation in the existing condition and for a possible range of pipe roughness values in the clean condition (Manning's roughness from 0.015 to 0.013). The representative inflow rates noted on the schematic are approximate values for the typical inflows from upstream sewersheds in a large wet weather event; these values are for conceptual reference. Actual inflow hydrographs provided by the technical program manager were used for the model simulations.

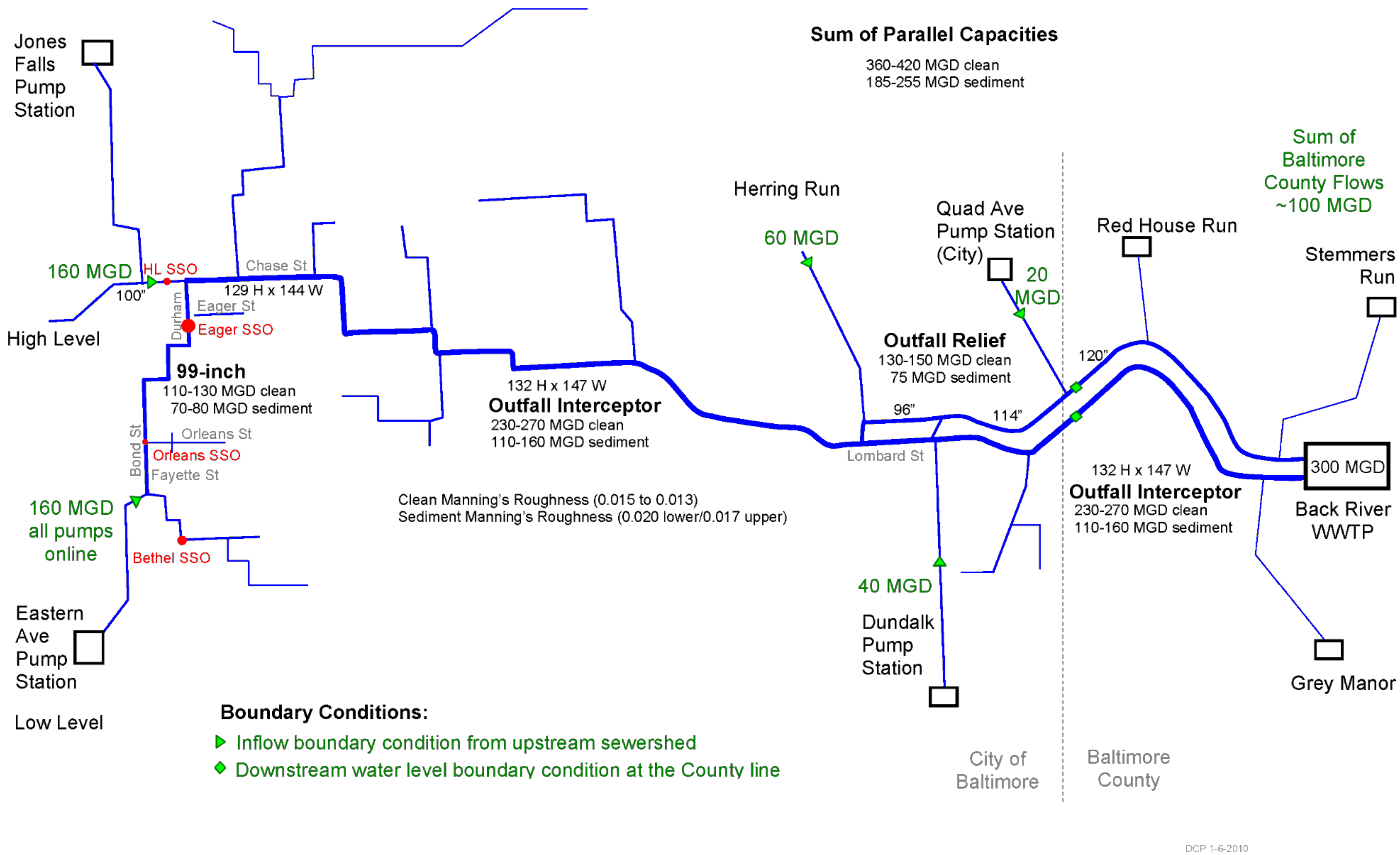


Figure 1.2 Schematic of Outfall Sewershed Trunk Sewers

1.3 Consent Decree Requirements

A Consent Decree (CD) was agreed upon between the City of Baltimore (City), the United states Environmental Protection Agency (USEPA) and the Maryland Department of the Environment (MDE), executed in April 2002 and issued May, 2002. As stipulated on page 22 of the CD, the City shall identify all components that cannot manage peak flows during a full range of storm events. The City shall then identify the required improvements necessary to ensure long term capacity with no sanitary sewer overflows (SSO) for the full range of storm events. These design storms include the three-month storm, having a duration equal to the time of concentration for the sewershed (5 hours) and the 1-, 2-, 5-, 10-, 15-, and 20-year, 24 hours storms.

1.4 Guidelines and Requirements

As specified in the Consent Decree, the future conditions model shall be used to determine the requirements necessary to convey all the flows without a sanitary sewer overflow (SSO). The future conditions model, as outlined in the *Baseline Analysis and Capacity Assessment Report*, dated October 2, 2009, projects the population to year 2025 and includes a 10% increase in average daily infiltration to account for pipe deterioration.

Per the CD and the City of Baltimore, the improvements Baltimore shall consider to assure adequate capacity shall include but not be limited to replacement of malfunctioning pumping station equipment, installation of pumping station back-up equipment, reduction of inflow and infiltration, installation of larger replacement sewers or relief sewers, sewer pressurization, and storage (both inline and offline).

1.5 Alternative Selection Process

The minimum design requirement for the City of Baltimore is the ability to convey at least a 2-year storm event. Based on this, the required upgrades to only convey the 3-month storm, and 1-year storm were not examined. The analysis begins with the required improvements necessary to convey the 2-year event without any SSOs. The 5-year, 10-year, 15-year and 20-year events were also evaluated.

1.5.1 Purpose of Alternatives Evaluation

The purpose of the alternatives evaluation was to determine the most feasible, cost effective method to improve the collection system to alleviate separate sewer overflows (SSO) within the Outfall Sewershed. The evaluation of each alternative was based on the following factors:

- Hydraulic performance at various sized wet weather events
- Constructability issues
- Cost effectiveness

1.5.2 Assumptions

In performing the hydraulic model simulations for the required design storm events, the following assumptions were made:

a. Year 2025 estimated average daily flow rates with diurnal peaking factors and a 10 percent increase of daily infiltration from baseline conditions.

b. The design storms apply rainfall hyetographs uniformly over the City and do not have spatial variations (only variations with time).

c. The design storms are NRCS-NOAA rainfall distribution, which are synthetic distributions representing rainfall conditions that have a uniform return period for a wide range of rainfall durations. For example, the 10-year design storm has a 10-year return period for a 1 hour duration as well as 6, 12, and 24-hour durations.

d. Wet weather flows in Info-Works are based on a median capture coefficient (R). Winter storms typically generate higher R values than summer storms. This is because in the winter the ground water table is higher; hence, more rainfall ends up in the sanitary sewer. In the summer conditions reverse. Dry soils and surface evaporation result in less rainfall finding its way to the sanitary pipes as the ground water table is lower due to evaporation, and a greater withdrawal by vegetation. The model does not account for antecedent moisture conditions and is not well suited for the analysis of back to back storms.

1.5.3 Screening of Initial Alternatives

The results of the hydraulic modeling highlighted two major areas of concern that contribute to SSOs in the existing sewer system in the Outfall Sewershed.

- One area is located where the Eastern Avenue Pump Station discharges into the 99-inch sewer, near the intersection of Fayette and Bond Streets (Fayette/Bond). The 99-inch sewer has a non-surcharged capacity of approximately 120 mgd. The Eastern Avenue Pump Station is able to discharge up to 160 mgd which exceeds the conveyance capacity of the 99-inch sewer and creates a risk of overflows.

- The other critical area is near the intersection of Chase and Durham Streets where flows from upstream sewersheds discharge into the 129 x 144-inch Outfall Interceptor.

There were several alternatives initially considered to respond to these areas of concern. These alternatives were scrutinized during a preliminary screening process that was performed to decide which of the alternatives appeared to be feasible to solve the problem. Four alternative concepts were retained for further evaluation. The others, which did not appear to be feasible, were rejected from further consideration. Factors considered in the screening process included: whether or not the facilities considered would resolve the SSO issue identified in the hydraulic modeling, the size of the facilities, the constructability of the facilities and whether another alternative would provide the required relief in a more reasonable way.

A. Storage Alternatives. In general two storage facilities are needed, one at Chase/Durham and the other at Fayette/Bond, to provide the needed relief. Storage facilities and overflow weirs are sized to store the excess flow and prevent overflows. The storage tanks are dewatered after the event when there is available capacity in the conveyance and treatment systems.

B. Pump Station Alternatives. Alternatives were considered that included the use of a pump station and force main to relieve hydraulic pressure at the two locations identified above. Based on the topography in the area, it was determined that a pumping system was not needed to convey the flows downstream. The topography allowed gravity flow, for the most part. It was also discovered that the pumping system would need to be extremely large to handle the large wet weather peak flows. Furthermore, the size of the force main was almost as large as the size of the gravity sewers proposed to convey the excess flows to a downstream location. The pump station alternatives were rejected as not feasible at this point of the evaluation.

C. Conveyance Alternatives (without Storage). These alternatives consist of providing conveyance capacity, via a relief sewer, from the two critical locations in the Outfall Sewershed (Chase/Durham and Fayette/Bond) to the upstream end of the existing Outfall Relief Sewer (along Lombard Street). This type of alternative may not be viable unless the downstream boundary conditions are improved to the point that would allow the free discharge of the excess flow back into the Outfall Sewer system.

D. Conveyance Alternatives (with Storage). These alternatives consist of relief sewers (similar to the alternatives described in Section C above) that are sized to provide inline storage along with conveying the excess flow downstream to the existing Outfall Relief sewer. The alternative relief sewers (most likely in the form of tunnels) provide storage (based on the size of storm event) to allow the excess flow to remain in the new relief sewer until there is capacity in the existing sewer system.

E. WWTP Alternatives. Alternative improvements at the WWTP are evaluated under the System-Wide Flow Evaluation portion of the project.

F. Infiltration and Inflow (I/I) Reduction. I/I reduction alternatives are evaluated under the System-Wide Flow Evaluation portion of the project.

Table 1.1 is a summary of the initial alternative concepts considered for evaluation. The screening results identify the concepts that were rejected, those that were retained for further evaluation, and those that are to be evaluated by others.

Table 1.1 Summary of Initial Alternative Concepts Considered for Evaluation

Alternatives	Screening Results
A. STORAGE	
1. Storage facility at upstream end of 99-inch sewer	Retained for further Evaluation
2. Storage at upstream end of Outfall Interceptor	Rejected
3. Two Storage Facilities (Alts.1.a & 1.b Combined)	Retained for further Evaluation
B. PUMP STATION (With Force Main)	
1. Pump station at Chase and Durham w/ force main to the upstream end of the existing Outfall Relief sewer	Rejected
2. Pump station at Fayette and Bond w/ force main to the upstream end of the existing Outfall Relief sewer	Rejected
C. CONVEYANCE (Without Storage)	
1. Relief sewer along Outfall Interceptor	Retained for further Evaluation
2. Relief sewer along Fayette Street	Retained for further Evaluation
D. CONVEYANCE/STORAGE	
1. Relief sewer tunnel along Outfall Interceptor (w/ downstream storage)	Rejected
2. Relief sewer along Fayette St. (w/ downstream storage)	Rejected
3. Relief sewer along Outfall Interceptor (w/ inline storage)	Retained for Further Evaluation
4. Relief sewer along Fayette Street (w/ inline storage)	Retained for Further Evaluation
E. WWTP ALTERNATIVES	
1. Storage at WWTP	Evaluated by Others
2. High Rate Treatment	Evaluated by Others
3. Adjustable Weir at WWTP	Evaluated by Others
4. Upgrade WWTP – To Achieve Higher Flow Capacity	Evaluated by Others
F. SYSTEM-WIDE FLOW MANAGEMENT	
1. Infiltration and Inflow Reduction	Evaluated by Others

Two general alternative concepts were retained for further consideration and evaluation at each of the two sites:

- Two storage tank facilities, one located at each of the critical locations in the Outfall Sewershed.
- A single tunnel facility that provides inline storage volume and conveys flow to a downstream location in the system where the Outfall Relief sewer is parallel to the Outfall Interceptor.

2.0 Baseline Analysis and Capacity Assessment (BACA): Re-evaluation with Revised Future 2025 Boundary Conditions

2.1 Previous BACA Evaluation

The Baseline Analysis and Capacity Assessment (BACA dated October 2, 2009) identified locations of overflows in the Outfall Sewershed and presented causes of those SSOs. The report addressed the risk of overflows for Baseline (year 2007) and Future 2025 conditions.

At the time of writing the BACA report, the boundary conditions to be applied to the Outfall Sewershed model for the Future 2025 conditions did not reflect recommended improvements to facilities in upstream sewersheds that had the potential to increase flows to the Outfall Sewershed. Subsequent to that time, the Future 2025 boundary conditions were refined two times by the Technical Program Manager to reflect the recommended upstream improvements. The revised boundary conditions, referred to in this report as “Upstream Improvements” boundary conditions, are more severe than those used in the simulations that are presented in the BACA report. Inflow hydrographs from the Low Level and High Level sewersheds are significantly larger (in volume and peak flow rate) and the downstream level boundary conditions at the County Line are significantly higher.

For example, in the 10-year design storm event, the peak flow from the Low Level Sewershed (from the Eastern Avenue Pump Station) increased approximately 70 MGD and the peak flow from the High Level Sewershed increased approximately 50 MGD. These higher flows increased the downstream level boundary condition at the County Line from a 1 foot surcharge to a 4 foot surcharge above pipe crown.

The Future 2025 results in the original BACA report are useful in that they identify locations with a SSO risk and sections of pipes that have hydraulic restrictions. The qualitative results are informative, but the numerical magnitude of overflow volumes and peak overflow rates in the original BACA report for the Future 2025 condition are based on the original boundary conditions which produce significantly smaller simulated overflows. Consequently, this Alternatives Analysis report is based on an analysis using the Upstream Improvements boundary conditions, presented in the following section.

2.2 Upstream Improvements Results

Future 2025 model simulation results are based on the boundary conditions that reflect recommended improvements to resolve SSOs in upstream sewersheds during the 2, 5, 10, 15, and 20-year design storm events. Table 2.1 presents the simulated overflow volumes in the Outfall Sewershed. This table is to be compared to Table 5.3.3A in the BACA report. Figure 2.1 is a bar graph of the sum of simulated SSO volumes in the Outfall Sewershed model for the various return period design storms. The first bars labeled “BACA” are the results presented in the BACA report. The second bars labeled “Upstream Improvements” are based on the most recent refinement of the boundary conditions that reflect all recommended upstream improvements and are used as the basis of this alternatives analysis.

The Upstream Improvements overflow volumes are the basis for determining the volume of overflow eliminated by the Outfall Sewershed alternatives. Subsequently, the unit cost of overflow volume elimination ($/gallon) was calculated using the cost of the alternative and the overflow volume eliminated relative to the Upstream Improvements simulation results.

Table 2.2 contains the simulated peak SSO discharge rates and is to be compared to Table 5.3.3B in the BACA report. Figure 5.3.1 from the BACA report is reproduced here for reference; this figure shows the Year 2025 Conditions, Hydraulic Restrictions Return Period Analysis. This figure identifies simulated SSO locations and the sections of pipes that are surcharged. Not all surcharged pipes are coincident with hydraulic restrictions. The figure from the BACA report identifies the sections of pipes that are both surcharged and a hydraulic restriction with a bold line. This figure contains a great deal of information. In the appendix of the BACA report this figure and others like it are reproduced in a larger format and with subsections of the figures shown again with zoomed in views to improve the clarity of the images.

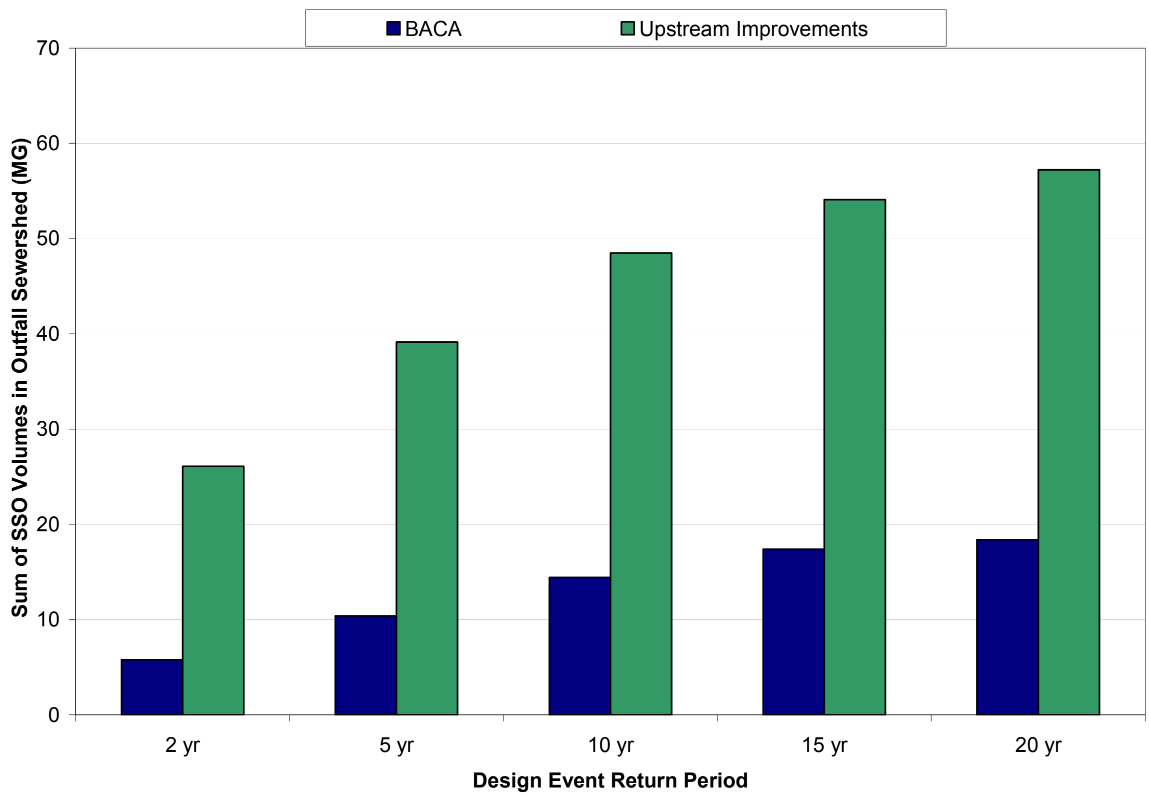


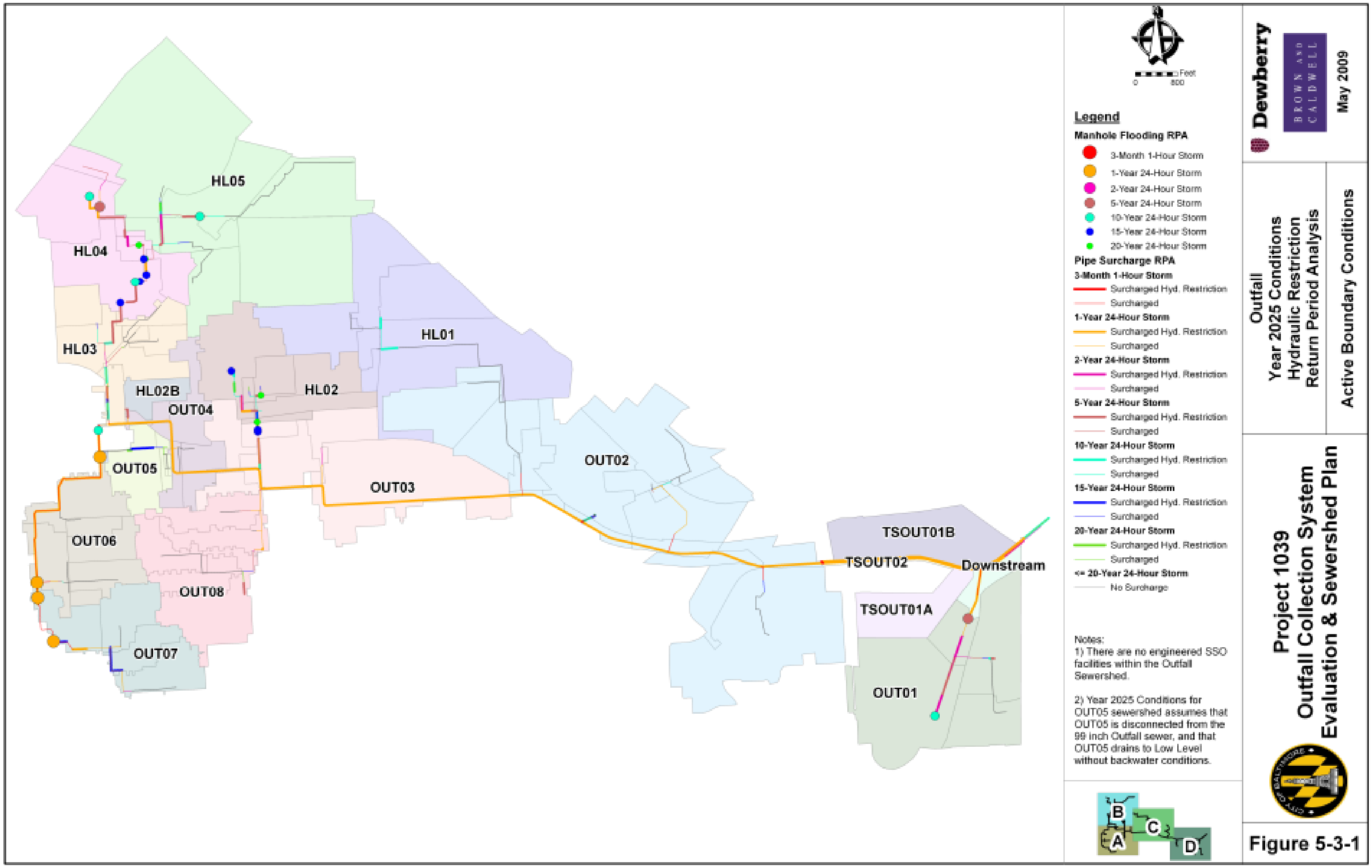
Figure 2.1 Sum of Simulated SSO Volumes in the Outfall Sewershed Showing Increase Due to Revised Boundary Conditions with Upstream Improvements

Table 2.1 SSO Volume – Future 2025 Flooding Return Period Analysis – Upstream Improvements Conditions							
Manhole	**2-yr**	**5-yr**	**10-yr**	**15-yr**	**20-yr**	**Meter Basin**	**Location**
S45CC_007MH	23.137	33.544	40.447	44.689	46.721	OUT06	Durham Street, south of Eager Street
S45CC_021MH	-	-	-	-	-	OUT05	Eager Street, at Durham Street (Future: Disconnected from Outfall)
S43E__016MH	1.487	2.115	2.595	2.914	3.125	OUT07	Bethel Street and Moyer Street
S43A__038MH	1.275	2.742	3.851	4.481	4.926	OUT06	Bond Street, at Orleans Street
S43C__022MH	0.206	0.741	1.411	1.683	1.983	OUT06	Bond Street, between Orleans Street and Fayette Street
S69C__002MH	0.000	0.003	0.095	0.145	0.189	OUT01	Sewer along RR tracks parallel to and between Kane St and Interstate 95. Behind the City of Baltimore Solid Waste Station at 111 Kane St.
S45OO_014MH	0.000	0.010	0.037	0.050	0.061	HL04	Wolfe Street at Darley Avenue
S69G__005MH	0.000	0.000	0.025	0.053	0.084	OUT01	Railroad tracks between Kane St and Interstate 95, at Eastern Ave.
S47MM_042MH	0.000	0.000	0.017	0.040	0.065	HL05	Sinclair Lane at Homestead Street
S43OO_002MH	0.000	0.000	0.001	0.006	0.012	HL04	Cliftview Avenue, half a block east of Wolfe Street
S45EE_015MH	-	-	-	-	-	near OUT06	Durham Street, south of Chase Street
S45KK_020MH	0.000	0.000	0.000	0.000	0.000	HL04	Lanvale Street, where the sewer turns south along Washington Street
S45KK_031MH	0.000	0.000	0.000	0.007	0.017	HL04	Lafayette Avenue, where the sewer turns south along Castle Street
S49EE_004MH	0.000	0.000	0.000	0.004	0.015	HL02	Luzerne Avenue, at Beryl Avenue
S45KK_026MH	0.000	0.000	0.000	0.002	0.005	HL04	Lafayette Avenue, between Chester Street and Castle Street
S45KK_003MH	0.000	0.000	0.000	0.001	0.005	HL04	Chester Street (west side of street), north of Lafayette Avenue
S49GG_039MH	0.000	0.000	0.000	0.001	0.008	HL02	Milton Avenue, north of Preston Street
S45MM_014MH	0.000	0.000	0.000	0.000	0.001	HL04	Chester Street (east side of street), south of North Avenue
S49EE_007MH	-	-	-	-	-	HL02	Luzerne Avenue, at Beryl Avenue
S49EE_029MH	0.000	0.000	0.000	0.000	0.001	HL02	Luzerne Avenue, between Beryl Avenue and Chase Street
S45MM_002MH	-	-	-	-	-	HL04	Alley parallel to North Avenue and E. 20th Street, between Castle Street and Chester Street
S45MM_018MH	-	-	-	-	-	HL04	Chester Street (west side of street), south of North Avenue
S49GG_032MH	-	-	-	-	-	HL02	Biddle Street, just east of Luzern Avenue
S43C__017MH	0.000	0.000	0.004	0.000	0.014	OUT07	just south of Fayette and Bond
S43C__026MH	0.000	0.000	0.003	0.000	0.011	OUT07	just south of Fayette and Bond
Sum of SSO (MG)	**26.1**	**39.2**	**48.5**	**54.1**	**57.2**		**Total for the Outfall Sewershed only**
S43EE_034MH	3.2	6.2	8.8	9.7	11.2	HL end	High Level Sewershed, Chase near Rutland, just upstream of the Outfall Interceptor
Sum of SSO (MG)	**29.3**	**45.4**	**57.3**	**63.8**	**68.5**		**Total including overflow in High Level at S43EE_034MH**

Note: Compare to Table 5.3.3A in BACA Report (October 2, 2009)

Table 2.2 Peak SSO Discharge Rate – Future 2025 Flooding Return Period Analysis – Upstream Improvements Boundary Conditions							
Manhole	**2-yr**	**5-yr**	**10-yr**	**15-yr**	**20-yr**	**Meter Basin**	**Location**
S45CC_007MH	103.77	118.84	126.26	129.42	128.16	OUT06	Durham Street, south of Eager Street
S45CC_021MH						OUT05	Eager Street, at Durham Street (Future: Disconnected from Outfall)
S43E__016MH	9.86	11.72	12.73	13.84	14.04	OUT07	Bethel Street and Moyer Street
S43A__038MH	27.08	28.28	38.61	38.91	39.13	OUT06	Bond Street, at Orleans Street
S43C__022MH	9.31	9.74	22.00	21.49	22.71	OUT06	Bond Street, between Orleans Street and Fayette Street
S69C__002MH	0.00	0.25	2.02	2.49	2.83	OUT01	Sewer along RR tracks parallel to and between Kane St and Interstate 95. Behind the City of Baltimore Solid Waste Station at 111 Kane St.
S45OO_014MH	0.00	0.54	1.13	1.29	1.39	HL04	Wolfe Street at Darley Avenue
S69G__005MH	0.00	0.00	0.97	1.58	2.15	OUT01	Railroad tracks between Kane St and Interstate 95, at Eastern Ave.
S47MM_042MH	0.00	0.00	0.51	0.92	1.23	HL05	Sinclair Lane at Homestead Street
S43OO_002MH	0.00	0.00	0.12	0.39	0.64	HL04	Cliftview Avenue, half a block east of Wolfe Street
S45EE_015MH						near OUT06	Durham Street, south of Chase Street
S45KK_020MH	0.00	0.00	0.00	0.00	0.02	HL04	Lanvale Street, where the sewer turns south along Washington Street
S45KK_031MH	0.00	0.00	0.00	0.45	0.78	HL04	Lafayette Avenue, where the sewer turns south along Castle Street
S49EE_004MH	0.00	0.00	0.00	0.28	0.62	HL02	Luzerne Avenue, at Beryl Avenue
S45KK_026MH	0.00	0.00	0.00	0.10	0.22	HL04	Lafayette Avenue, between Chester Street and Castle Street
S45KK_003MH	0.00	0.00	0.00	0.07	0.17	HL04	Chester Street (west side of street), north of Lafayette Avenue
S49GG_039MH	0.00	0.00	0.00	0.09	0.54	HL02	Milton Avenue, north of Preston Street
S45MM_014MH	0.00	0.00	0.00	0.00	0.05	HL04	Chester Street (east side of street), south of North Avenue
S49EE_007MH						HL02	Luzerne Avenue, at Beryl Avenue
S49EE_029MH	0.00	0.00	0.00	0.00	0.11	HL02	Luzerne Avenue, between Beryl Avenue and Chase Street
S45MM_002MH						HL04	Alley parallel to North Avenue and E. 20th Street, between Castle Street and Chester Street
S45MM_018MH						HL04	Chester Street (west side of street), south of North Avenue
S49GG_032MH						HL02	Biddle Street, just east of Luzern Avenue
S43C__017MH	0.00	0.00	0.32	0.00	0.34	OUT07	just south of Fayette and Bond
S43C__026MH	0.00	0.00	0.25	0.00	0.27	OUT07	just south of Fayette and Bond
S43EE_034MH	24.15	34.34	40.04	42.77	53.27	HL end	High Level Sewershed, Chase near Rutland, just upstream of the Outfall Interceptor

Note: Compare to Table 5.3.3B in BACA Report (October 2, 2009)



Reproduction of Figure 5-3-1 from the BACA Report

2.3 Description of Hydraulic Issues in the Outfall Sewershed

The largest overflows in the Outfall Sewershed are located on or near the 99-inch sewer and are caused by conveyance limitations of the large diameter trunk sewers and high inflows from upstream sewersheds. Smaller overflows occur along the smaller branch sewers due to localized hydraulic restrictions and high flows generated in the subcatchment areas. The following sections describe the hydraulic factors at work in the Outfall Sewershed that need to be addressed by potential alternatives. These hydraulic factors are discussed as background information which is useful in the development of alternatives strategies.

2.3.1 Source of flows

There are two types of flow in the Outfall Sewershed model. One type of flow is generated by rainfall on subcatchments within the Outfall Sewershed. The other type is inflow applied as a boundary condition to the Outfall Sewershed model originating from flow generated in upstream sewersheds (the Jones Falls, High Level, Low Level, Herring Run, and Dundalk sewersheds).

Inflows from upstream sewersheds are much larger than flows generated by subcatchments within the Outfall Sewershed. Therefore, most of the alternatives will focus on managing overflows caused by the high inflows from upstream (particularly Jones Falls, High Level, and Low Level). A few smaller alternatives will address overflows in the smaller branch sewers due to high wet weather flows in the subcatchments, but these overflows are relatively small compared to the overflows along the large trunk sewers due to the inflow boundary conditions.

Reduction of rainfall derived infiltration and inflow (RDII) is a possible means of reducing peak wet weather flows in general. However, because most of the flows into the Outfall Sewershed model are from upstream sewersheds, alternatives targeting SSOs along the trunk sewers do not assume RDII reduction in the upstream sewersheds beyond what is already represented in the Upstream Improvements boundary conditions. The inflow boundary conditions are those that were provided by Technical Program Manager (9-18-2009).

It is possible that RDII reduction may be useful as an alternative to mitigate some of the SSOs along the smaller branch sewers; this form of RDII reduction is briefly discussed in this report. However, RDII reduction in the Outfall Sewershed area is not capable of reducing the magnitude of the larger SSOs related to the trunk sewers, which would require extensive and successful RDII reduction in the upstream sewersheds. Investigation of RDII reduction on that scale is beyond the scope of the Outfall Alternatives Analysis Report.

2.3.2 Relationship to Downstream Facilities: Boundary Conditions at the Baltimore County Line

The downstream extent of the Outfall Sewershed model is the Baltimore County Line. Water level boundary conditions define the conditions in the Outfall Interceptor and the Outfall Relief Sewer at the County Line. These boundary conditions are meant to represent the hydraulic conditions further downstream in Baltimore County and at the Back River WWTP. The boundary conditions provided by the Technical Program Manager (9-18-2009) are characterized by peak water levels that surcharge the Outfall Interceptor and the Outfall Relief Sewer. The surcharge implies a backwater condition which causes higher water levels and lower velocities than normal flow conditions without a backwater constraint. (A hydraulic backwater condition does not mean a reversal in the direction of flow. The flow continues downstream but the water levels are elevated and the slope of the hydraulic grade line (HGL) is less than the slope of the pipe invert.)

The backwater condition at the Back River WWTP has two significant adverse effects on the performance of the large diameter trunk sewers (the Outfall Interceptor, the Outfall Relief sewer, and the 99-inch sewer that serves the Eastern Avenue Pump Station Force Main). First, the backwater condition limits the effective conveyance capacity of the Outfall Interceptor by limiting the maximum possible slope of the HGL. When overflows occur at the upstream end of the Outfall Interceptor the HGL can not be any steeper than the slope between the ground surface elevation at the overflow and the water level at the County Line. Second, low velocities due to the backwater condition cause sediment to accumulate in the large diameter trunk sewers. Sediment further reduces the conveyance capacity by reducing the cross section area of the pipes and increasing hydraulic roughness.

The primary findings in this report are based on the Upstream Improvements boundary conditions provided by the Technical Program Manager. The downstream level boundary condition is lowered in some simulations to reflect possible downstream improvements at the Back River WWTP. The additional conveyance capacity provided by a steeper HGL greatly reduces the size of the alternative facilities needed in the Outfall Sewershed.

It is beyond the scope of this report for the Outfall Sewershed to investigate the specific details associated with the downstream improvements. It is assumed that the improvements reduce or eliminate the WWTP-induced backwater condition. For the purpose of this study, the downstream improvements are represented in the Outfall Sewershed model as a downstream level boundary condition at the County Line that does not exceed 48 feet (above NAVD88 datum). At 48 feet the Outfall Interceptor and Outfall Relief sewer are approximately 90% full with the water level one foot below the crown of the pipe. Further investigation of alternatives of this type will require ongoing collaboration with the Technical Program Manager using the Macro Model.

2.3.3 Hydraulic Factors Related to the 99-inch Sewer

The 99-inch circular sewer conveys flow by gravity from the Eastern Avenue Pump Station force main to the upstream end of the Outfall Interceptor. The following is a list of hydraulic factors related to the 99-inch sewer:

- The peak pumping rate from the Eastern Avenue Pump Station, with all pumps online, exceeds the capacity of the 99-inch sewer. The peak pumping rate is approximately 160 MGD.
- The clean, full pipe capacity of the 99-inch sewer is approximately 110 to 130 MGD (depending on the assumed Manning's roughness value) if the pipe is clean of sediment and flowing freely (i.e. no backwater conditions). With 10 to 20 inches of sediment, the capacity is reduced to approximately 70 to 80 MGD.
- Pumping in excess of the capacity of the 99-inch sewer results in a steepening of the HGL in the already surcharged pipe. Excessive water levels at the upstream end of the 99-inch are relieved by reversing the flow in the 24-inch pipe that serves the OUT07 meter basin. The reverse flow is relieved by overflowing at Bethel and Moyer Streets (manhole S43E__016MH) where the ground surface is relatively low (ground cover over the pipe is approximately 5 feet).
- Excess pumping to the 99-inch sewer can also result in overflows at manholes on Bond Street at the upstream end of the 99-inch sewer between Orleans Street and Fayette Street.
- High water levels in the Outfall Interceptor further impede the effective conveyance capacity of the 99-inch sewer, but this is a secondary cause of overflows at the upstream end of the 99-inch sewer. Model simulations indicate that even if the level at the upstream of the Outfall Interceptor were to be lower, the exceptionally high pumping rates from the Eastern Avenue Pump Station would require relief at the upstream end of the 99-inch sewer near Fayette and Bond Streets.

2.3.4 Hydraulic Factors Related to the Outfall Interceptor

The Outfall Interceptor is a concrete arch sewer that is over 100 years old. For most of the 20,000 ft length (in the model to the County Line) the size of the pipe is 132 inches high by 147 inches wide (the upstream 4,000 feet are 129 inches high by 144 inches wide). The following is a list of hydraulic factors related to the Outfall Interceptor:

- Inflows from the High Level and Jones Falls sewersheds enter the upstream end of the Outfall Interceptor at Chase and Durham Streets. Peak inflows at this location range from 155 to 170 MGD. After the High Level/Jones Falls flows join with the flow from the 99-inch sewer; the total peak flow is potentially in the range of 330 MGD (if there were no upstream overflows to relieve some of the excess flow).
- The full pipe capacity of the Outfall Interceptor is approximately 230 to 270 MGD (depending on the assumed Manning's roughness value) if the pipe is clean of sediment and flowing freely. The sediment accumulation in the Outfall Interceptor ranges from 11 to 42 inches and is typically between 20 to 40 inches. The sediment reduces the cross section area of the pipe 15 to 30%. With 20 to 40 inches of sediment present in the sewer, the capacity is reduced to approximately 110 to 160 MGD.
- High flows from upstream sewersheds produce high water levels at the upstream end of the Outfall Interceptor in the vicinity of Chase and Durham Streets. Overflows occur at manholes near the downstream ends of the 99-inch sewer and the 100-inch High Level sewer. Manhole S45CC_007MH on the 99-inch sewer

at Durham and Eager Streets is the location of the largest SSO volume in the Outfall Sewershed. Even though this manhole is along the 99-inch sewer it is effectively providing relief to the Outfall Interceptor. The low ground level (approximately 4 ft. of ground cover over the pipe) limits the maximum possible slope of the HGL along the Outfall Interceptor, thus limiting the maximum conveyance capacity of pipe.

- High downstream boundary condition water levels at the County Line further constrain the maximum possible slope of the HGL along the Outfall Interceptor and diminish the effective conveyance capacity of the Interceptor. However, overflows start to occur at Durham Street before the Outfall Interceptor surcharges at the County Line because of high upstream flow rates.
- Downstream improvements at the Back River WWTP can potentially lower the HGL at the plant and accommodate a steeper HGL along the Outfall Interceptor, steeper than the nominal invert slope. This drawdown effect can increase the conveyance capacity. With a drawdown, the capacity would be in the range of 280 to 330 MGD.

2.3.5 Hydraulic Factors Related to the Outfall Relief Sewer

The Outfall Relief sewer is a circular pipe that starts at the end of the Herring Run Siphon (near 6000 E. Lombard Street) and runs parallel to the Outfall Interceptor to the Back River WWTP. The Relief sewer diameter is initially 96 inches at the Herring Run Siphon, increasing to 114 inches at the Dundalk connection, and later 120 inches near the County Line. The following is a list of hydraulic factors related to the Outfall Relief sewer:

- The Outfall Relief sewer has a clean pipe capacity of approximately 130 to 150 MGD. Sediment (approximately 30 inches deep) in the Relief sewer reduces the capacity to approximately 75 MGD, assuming free flowing conditions.
- There are two interconnections (junction chambers) between the Outfall Relief sewer and the Outfall Interceptor. Other interconnections are downstream of the County Line, beyond the extent of the Outfall Sewershed model. These interconnections allow flow to be shared between the two conduits so that the water levels in both conduits are essentially equal.
- The sum of the full pipe capacities of the Outfall Interceptor and the Outfall Relief sewer is approximately 360 to 420 MGD if cleaned of sediment, and 200 MGD with sediment left in place.

3.0 Alternatives Analysis: Strategies and Evaluation

3.1 General Strategies and Evaluation Criteria

Several factors were considered during the evaluation process to determine the most feasible and cost effective alternative to recommend. The evaluation was based primarily on hydraulic factors. Consideration was given to constructability factors such as space to construct the facilities, depth of construction (geotechnical) and the disruption of local establishments and utilities. Construction costs were used to rate the alternatives to reach a recommended alternative.

3.1.1 Hydraulic

The InfoWorksCS® hydraulic model was used to evaluate the relief requirements in the Outfall Sewershed for the 2, 5, 10, 15 and 20-year design storms. The simulation results defined the required storage volumes and the peak flow rates needed to provide adequate relief to prevent SSOs for each design storm. The largest overflows occur near the 99-inch sewer when peak flow rates exceed the conveyance capacities of the large diameter trunk sewers. Smaller SSOs on the branch sewers were also addressed, but the primary alternatives evaluation is focused on the elimination of the trunk sewer SSOs.

3.1.2 Constructability

The alternatives were evaluated based on the type of construction anticipated, the availability of construction sites and the constructability of the facilities. Constructability evaluation included, among other things, the depth of excavation, and the disruption that construction would cause.

3.1.3 Costs

Construction costs were developed for all alternatives evaluated. To develop the estimated costs of construction, standard unit costs for sewer point repairs, sewer lining, sewer replacement, sewer cleaning, and manhole rehabilitation/replacement were provided by the City in 2008 dollars. The construction costs provided were fully loaded costs to address such items as mobilization, maintenance of traffic, paving restoration, bypass pumping and miscellaneous (non-sanitary) utility work. For costs not provided by the City (large diameter tunnels and pumping stations) recent projects within the City and surrounding areas were reviewed to assist in estimating the most probable fully loaded cost of construction. In addition, an independent cost estimate was performed by a third party estimator. The results are presented in Appendix A and used as the basis for the cost estimates given in Chapter 4.

In addition to these construction costs, an additional 42 percent was added to accommodate engineering design, construction management/inspection, administration, post-award engineering services and contingencies. A 7 percent annual inflation rate is used to project costs for years beyond 2008.

The cost tables, assumptions, and other information used to calculate the various cost values are presented in Appendix A. The cost calculations were based on the following assumptions:

- For small storage tank construction, $6 per gallon was used as the basis for costs for branch sewer storage alternative, including all costs for pumpout and other requirements. Large storage tank costs are presented in Appendix A.
- Tunnel construction costs are presented in Appendix A. Construction costs provided by the City of Baltimore, in the BASES Manual are targeted for the rehabilitation and replacement of smaller sewer sizes than required for the Outfall Sewershed alternatives. Therefore, the cost data presented in Appendix A were developed specifically for larger facilities and based on historical data comparable to conditions in the City of Baltimore.
- For pump station construction, the costs are based on the information presented in Appendix A. The cost data was developed specifically based on City of Baltimore historical data.
- The construction costs were adjusted by adding engineering, construction management and contingency costs to arrive at the total estimated cost values presented herein.

Estimated costs for sediment removal were used to determine the cost effectiveness of removing the sediment from the outfall sewer system versus building larger facilities to compensate for the reduced pipe capacities due to the presence of the sediment.

3.2 Hydraulic Evaluation: Branch Sewer Alternatives

Overflows on the branch sewers are relatively small in volume compared to overflows along the trunk sewers. Solutions include increasing pipe sizes, small storage facilities, and sewer rehabilitation to reduce infiltration and inflow (RDII). Brief descriptions of alternatives for the branch sewers will be presented next before addressing alternatives to resolve the larger SSOs along the trunk sewers.

There are no simulated overflows in the branch sewers of the Outfall Sewershed until the 5-year return period event, which produces a small overflow in the OUT01 meter basin and a very small overflow in the HL04 meter basin. The 10-year event produces larger overflow volumes in the HL04 and OUT01 meter basins and also activates a SSO in the HL05 meter basin. The 15-year event activates a small overflow in the HL02 meter basin.

3.2.1 HL04 Alternatives

Peak flows surcharge the sewers for the entire length of meterbasins HL03 and HL04 from the upstream end (north of Sinclair Lane) to the downstream connection at the Outfall Interceptor (at Wolfe Street and Chase Street). There is a risk of SSOs at several locations along this sewer system where the maximum HGL approaches the ground surface. Overflows are most likely at manhole S45OO_014MH (Wolfe Street and Darley Street) because of a low ground surface elevation at this point (less than 4 feet of cover). The SSO location is active for the 5-year and larger events.

Possible solutions include sealing the manhole, raising the manhole rim to an elevation that is similar to neighboring manholes (approximately 3 feet), building a small storage tank, or rehabilitation of sewers in the Darley/Cliftview Avenue neighborhood to reduce I/I. A storage tank alternative or sewer rehabilitation to reduce RDII will reduce peak flows to the downstream pipes leading to the Outfall Interceptor, thus decreasing the risk of SSO at other locations which do not have simulated SSOs but are at risk of SSOs due to high water levels.

In the 15 and 20-year events, another storage facility is needed in the vicinity of North Avenue and Chester Street to reduce peak flows to the downstream sections of pipe. Not only do the larger events require additional storage at the Wolfe and Darley location, but 554 LF of pipe along Wolfe Street and Darley Street need to be upsized from 10 to 12 inches. Required sizes of alternative facilities are listed in Table 3.1 for the various return period events that cause overflows in the HL04 meter basin.

Table 3.1 Alternative Facilities to Eliminate SSOs in the HL04 Meter Basin						
Site	**Facility**	**2-yr**	**5-yr**	**10-yr**	**15-yr**	**20-yr**
HL04 Wolfe & Darley	Storage Tank Volume (MG)	None	0.047	0.065	0.058	0.074
HL04 North & Chester	Storage Tank Volume (MG)	None	None	None	0.073	0.107
HL04 Darley Street to Sinclair Street	Replacement Pipe Diameter (inches) Length (LF)	None	None	None	12” 554 LF	12” 554 LF

If a RDII reduction alternative were to be used instead of a storage tank, the peak flows would need to be successfully reduced 30 to 50% from the Darley/Cliftview Avenue neighborhood to eliminate the overflow at the Wolfe and Darley location. More extensive RDII reduction would be needed to provide the same benefit at the North and Chester storage tank.

The cost of RDII reduction was investigated. The Darley/Cliftview neighborhood has approximately 11,000 LF of sewers ranging in size from 8 to 24 inches. The cost to rehabilitate these sewers to reduce RDII would be approximately $3 million.

RDII reduction in the sewers upstream of the North/Chester overflow location would require rehabilitation of approximately 20,000 LF of pipe with a cost of $5.5 million. The total cost of RDII in the HL04 meter basin area would be approximately $8.5 million.

For comparison, the costs of the alternatives listed in Table 3.1 are approximately $1 million for the 20-year event and less for the smaller events. (These costs are itemized in Chapter 4). Thus the costs of the alternatives listed in Table 3.1 are approximately an order of magnitude less than the cost of RDII reduction.

The 46 acre subcatchment area representing the Darley/Cliftview neighborhood is approximately one half mile upstream of the HL04 meter location. Meter basin HL05 and several other subcatchments in meter basin HL04 contribute flow to the HL04 meter location. HL04 has a total area of 502 acres. Consequently, the flow generated by the Darley/Cliftview neighborhood subcatchment accounts for less than 10% of the flow calibrated to the HL04 meter site. Site specific monitoring at this location near the upstream end of the meter basin is recommended to further refine the risk of overflows at the Wolfe and Darley site.

3.2.2 HL05 Alternatives

In the HL05 meterbasin, there is a simulated SSO along the 12-inch sewer along Sinclair Lane at Homestead Street (manhole S47MM_042MH) for the 10-year and larger events. With approximately 6 feet of ground cover, this manhole is vulnerable to overflow because of a downstream hydraulic restriction along Collington Avenue. At Sinclair Lane and Collington Avenue, the flow in the 12-inch sewer is joined by flow from a 10-inch sewer from the north (serving the Clifton Park/Heritage High School area). The 12-inch pipe along Collington Avenue downstream of this junction is a hydraulic restriction until the size increases to 15-inches near North Avenue.

Increasing the size of the pipe along Collington Avenue from 12 to 15-inches is necessary to eliminate the SSO further upstream at Sinclair and Homestead. The 15-inch replacement pipe would run 592 LF along Collington Avenue from manhole S47MM_031MH (Sinclair & Collington) to manhole S45MM_025MH (in an alley west of Collington Avenue and north of North Avenue).

For the 15 and 20-year events the 12-inch sewer along Sinclair Lane also needs to be upsized to 15-inches. This segment is 751 LF from manhole S47MM_042MH (Sinclair and Homestead) to Collington Avenue at manhole S47MM_031MH.

Table 3.2 summarizes the facilities to eliminate SSOs in the HL05 meter basin for each design storm.

Table 3.2 Alternative Facilities to Eliminate SSOs in the HL05 Meter Basin

Site	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
HL05 Collington Ave	Replacement Pipe Diameter (inches) Length (LF)	None	None	15” 592 LF	15” 592 LF	15” 592 LF
HL05 Sinclair Ln	Replacement Pipe Diameter (inches) Length (LF)	None	None	None	15” 751 LF	15” 751 LF

3.2.3 OUT01 Alternatives

The 18-inch sewer serving meterbasin OUT01 runs along the railroad tracks parallel to and between Kane Street and the Interstate-95 freeway. This branch sewer has two sections, an upstream section at a higher elevation and a downstream section at a lower elevation. There is one simulated SSO location in the lower section for the 5-year and larger events, and one simulated SSO at the upstream end of the upper section for the 10-year and larger events.

The two sections are connected by a steep segment of pipe that allows the flow to descend rapidly to the lower section at manhole S69C__002MH, which is the site of the OUT01 flow meter. The ground level at this manhole is approximately 10 feet above the crown of the pipe. Even though there is adequate ground cover, the rim of this manhole is approximately 7 feet lower than the adjacent manhole rim and is the first point to experience overflows along this branch sewer. The simulated SSO, occurring for the 5-year and larger events, is caused by high peak flows that exceed pipe capacity. The volume of the SSO increases when the Outfall Interceptor is surcharged, but this downstream surcharge condition is not the primary cause of the SSO. Increasing the pipe size from the overflow site to the connection with the Outfall Interceptor eliminates the overflow. The length of the sewer replacement is 1012 LF from manhole S69C__002MH to the connection to the Outfall Interceptor at manhole S71A__007MH.

Manhole S69G__005MH is the upstream end of the upper section in the model. This manhole, at Eastern Avenue, is the location of a small simulated overflow for the 10-year and larger events. The first pipe section in the model is a 15-inch pipe; all of the other pipe sections along this branch sewer are 18-inch diameter. The 10-year event requires the replacement pipe of approximately 400 LF of pipe from manhole S69G__005MH to the next manhole north, S69G__008MH. The replacement pipe is upsized from 15 to 18-inches. The 15 and 20-year events require 1600 LF of pipe upsized to 21 inches from manhole S69G__005MH to manhole S69E__005MH.

Table 3.3 summarizes the facilities to eliminate SSOs in the OUT01 meter basin for each design storm.

Table 3.3 Alternative Facilities to Eliminate SSOs in the OUT01 Meter Basin

Site	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
OUT01 Upper Section	Replacement Pipe Diameter (inches) Length (LF)	None	None	18” 400 LF	21” 1600 LF	21” 1600 LF
OUT01 Lower Section	Replacement Pipe Diameter (inches) Length (LF)	None	24” 1012 LF	24” 1012 LF	24” 1012 LF	24” 1012 LF

Additional monitoring of flow in the OUT01 sewer is recommended to determine if actual flows exceed the pipe capacity for larger and more intense rainfall events. During the brief monitoring period, the largest observed flow was 1.8 MGD in response to rainfall on 4/15/2007 with a peak intensity of 0.36 inches/hour. This observed peak flow is much less than the full pipe capacity of 3.7 MGD. The observed peak flow is also much less than the simulated peak flow in the model (which is 6.4 MGD in the 5-year design storm) that produces the simulated overflow. Additional monitoring during a wet weather period with a rainfall intensity of the same order of magnitude as the design storm is needed to verify whether upsizing of the pipes is needed.

3.2.4 HL02 Alternatives

Simulated overflows in meter basin HL02 occur in the 15 and 20-year events at several locations along Luzerne Street and Milton Street because of high wet weather flows and limited conveyance capacity. Surcharging in the Outfall Interceptor also contributes to the overflows. The trunk sewer alternatives are effective in reducing the water level in the Outfall Interceptor for the 15-year event, such that alternative conveyance projects are not needed in the HL02 meter basin until the 20-year event.

In the 20-year event, high peak flow rates cause surcharging all along the length of the HL02 branch sewer. To eliminate the SSO, upsizing the pipe near the downstream end of the branch sewer is recommended; however, the sewer alignment crosses under the railroad tracks in this area. Therefore, it is recommended to upsize the pipe just north of and just south of the railroad tracks. The recommendation does not replace the pipes under the tracks. The replacement upsizes the pipes along Luzerne Street from 15 inches to 24 inches. Just north of the railroad, the replacement runs 134 LF from manhole S49EE_004MH (Beryl Street) to manhole S49EE_021MH. Just south of the railroad the replacement runs 137 LF from manhole S49CC_021MH to manhole S49CC_075UN (Ashland Street at the connection to the Outfall Interceptor). The total length of replacement along Luzerne Street is approximately 271 LF.

At the upstream end of the model there is also a small overflow at Milton Street north of Preston Street (manhole S49GG_039MH) in the 20-year event. The short 10-inch sewer that crosses under the road needs to be upsized to 15 inches for 46 LF from manhole S49GG_039MH to manhole S49GG_027MH.

Table 3.4 summarizes the facilities to eliminate SSOs in the HL02 meter basin for each design storm.

Table 3.4 Alternative Facilities to Eliminate SSOs in the HL02 Meter Basin

Site	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
HL02 Luzern St	Replacement Pipe Diameter (inches) Length (LF)	None	None	None	None	24” 271 LF
HL02 Milton St	Replacement Pipe Diameter (inches) Length (LF)	None	None	None	None	15” 46 LF

3.3 Description of Trunk Sewer Alternatives

It is assumed that sediment is removed from the trunk sewer in all of the alternatives presented below. When sediment is removed, the roughness is also assumed to be reduced. Initially, the Manning’s roughness coefficient (n) is assumed to be 0.013. However, because the results are very sensitive to this assumption, the system performance is also evaluated for a Manning’s roughness value of 0.015 to determine the necessary facilities to perform adequately for sub-optimum conditions.

Two key points in the system are vulnerable to overflow; one key point is at the upstream end of the Outfall Interceptor and the other is at the upstream end of the 99-inch sewer. Construction of an overflow weir at each key point is the direct approach to providing the needed relief. Alternative 1 uses two overflows weirs and two storage tanks. It is also possible to protect both key points with a single weir at the upstream end of the 99-inch sewer; this option is investigated as Alternatives 2 and 3. Section 3.3 describes the alternatives in general; whereas, Section 3.4 describes each of them in detail, including pipe diameters and lengths, storage tank volumes, and tunnel diameters.

3.3.1 Alternative 1: Storage Using Two Tanks

Alternative 1 uses two storage tanks to store excess flow and prevent SSOs as shown on Figure 3.1. An overflow weir at the upstream end of the 99-inch sewer is needed in the vicinity of Bond and Fayette Streets. This relief facility is called the Fayette weir in the discussion below. The facility should be located between Fayette and Orleans Streets, in close proximity to the connection from the Eastern Avenue Pump Station force main. The purpose of the Fayette weir is to limit the maximum water level at the upstream end of the 99-inch sewer to approximately 58 feet; at this level the 99-inch sewer is surcharged 3 feet and the risk of a SSO further upstream along the 24-inch branch sewer at Bethel Street and Moyer Street (manhole S43E__016MH) is minimized. The length of the Fayette weir is assumed to be 50 feet in the alternatives below. An important design parameter is that the weir should have adequate capacity to convey the excess peak flow from the Eastern Avenue Pump Station which is approximately 60 MGD.

Relief is also needed to protect the upstream end of the Outfall Interceptor from excessive surcharging in the vicinity of Chase and Durham Streets. The Chase weir may be located anywhere in the vicinity of Chase and Durham (either on the Outfall Interceptor itself or at the downstream end of the 99-inch sewer or the downstream end of the 100-inch High Level sewer). The purpose of the Chase weir is to limit the maximum water level at the upstream end of the Outfall Interceptor to no more than 57 feet; at this level the Outfall Interceptor is surcharged 3 feet and the risk of an SSO is minimized at Durham and Eager Streets (manhole S45CC_007MH). The Chase weir should be relatively long to allow significant overflow rates (into a storage tank) with a relatively small head on the weir. The alternatives assume a 50 foot long weir crest which is sufficient to pass approximately 100 MGD over the weir with 1 foot of head above the weir crest.

The two storage tanks attenuate the peaks of the inflow hydrographs so that peak flows are within the capacities of the large diameter trunk sewers assuming that the sediment has been removed. Alternative 1 assumes that there are no changes downstream at the Back River WWTP; consequently, the Outfall Interceptor is surcharged at the County Line. Without improvements at the Back River WWTP, the tanks in this alternative are sized to store the excess flow that can not be conveyed and treated.

After a wet weather event, the storage tanks are dewatered by lift stations which pump the excess volume back into the conveyance system at the same locations as the relief weirs. A one day dewatering period is used to size the pump capacities to minimize the opportunity for septic conditions to be generated in the tanks.

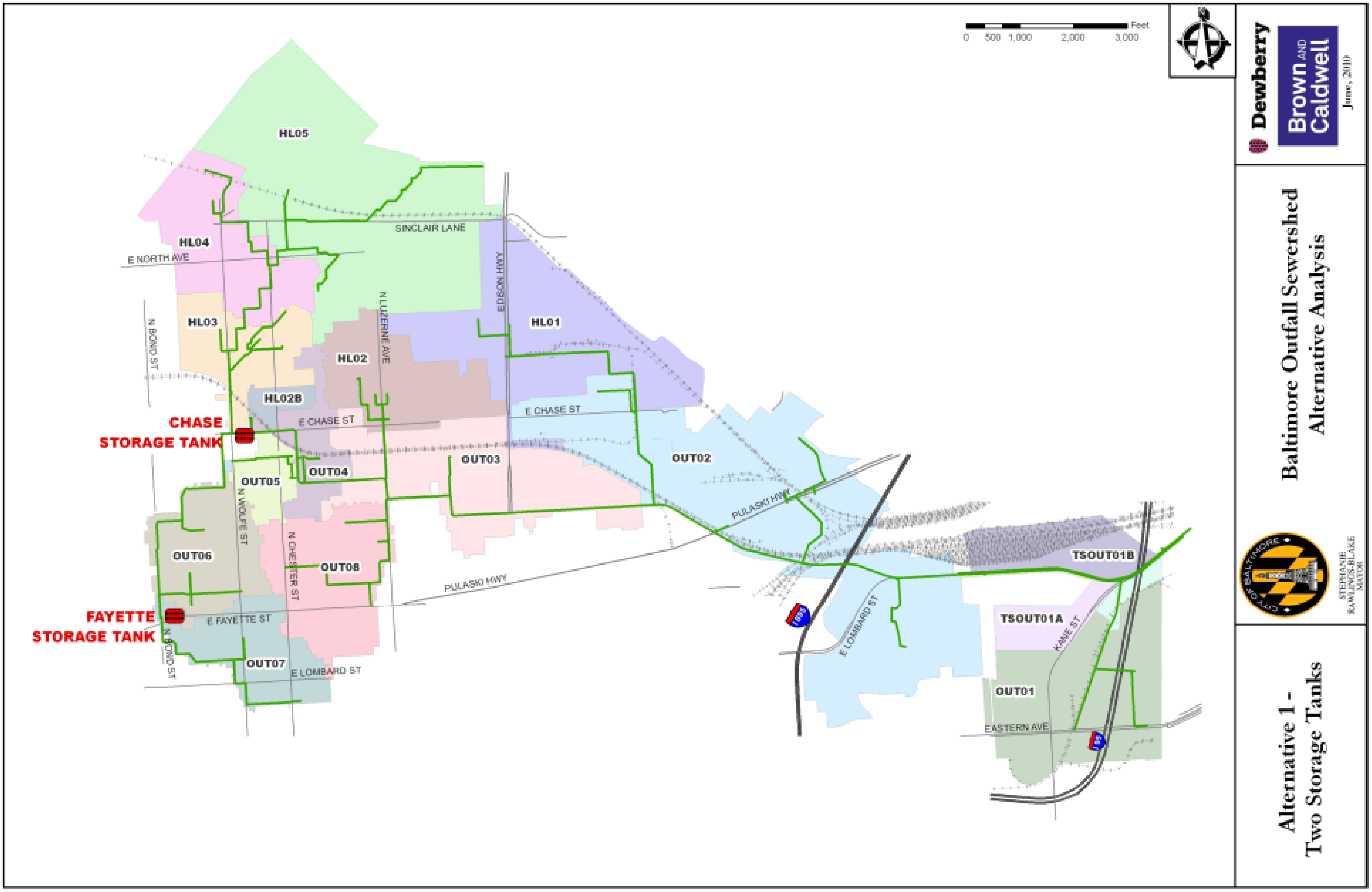


Figure 3.1 Alternative 1 Facilities: Two Storage Tanks

3.3.2 Alternative 2: Storage using One Tank, Assuming Downstream Improvements

Alternative 2 assumes that sediment is removed and downstream improvements at the Back River WWTP accommodate higher flow rates to the plant. The additional conveyance and treatment capacity downstream results in lower water levels at the County Line. This alternative demonstrates the significant improvement that can be achieved in system performance due to downstream improvements. Assuming that the cleaned pipes have a Manning's roughness value of 0.013, the additional conveyance in the Outfall Interceptor is sufficient to manage the 2-year event without simulated overflows. No new storage at either the Chase or Fayette weir sites is required for the 2-year event. In the larger events, only one storage tank at the Fayette weir location is necessary for the 5, 10, 15, and 20-year recurrence interval storms.

3.3.3 Alternative 3: Storage-Conveyance Tunnel, Assuming Downstream Improvements

Alternative 3 uses a tunnel instead of a storage tank to protect against overflows. The tunnel starts at the Fayette weir location and generally runs parallel to the Outfall Interceptor. The flow in the tunnel re-enters the Outfall Interceptor along Lombard Street where the Outfall Relief sewer runs parallel to the Outfall Interceptor. The tunnel connection is near the location where the Dundalk sewer connects to the Outfall Interceptor. Initially, the tunnel provides inline storage volume. After filling and surcharging, the tunnel flows like an inverted siphon to convey flow to the downstream connection point. The tunnel can be seen as an upstream extension of the Outfall Relief sewer. Instead of running immediately parallel to the Outfall Interceptor, the tunnel extends the relief directly to the Fayette Weir location where relief is needed to protect the 99-inch sewer from high pumping rates from the Eastern Avenue Pump Station. By diverting excess flow into the tunnel at the Fayette weir, both the 99-inch sewer and the Outfall Interceptor are protected from overflows. Figure 3.2 is a general sketch of the tunnel concept in Alternative 3. The actual route of the tunnel would be determined in further engineering design efforts.

Key assumptions and features of Alternative 3 are:

- Sediment is removed from the 99-inch sewer, the Outfall Interceptor, and the Outfall Relief sewer.
- Because sediment is removed, the roughness of the large diameter trunk sewers is reduced from the calibrated value (Manning's n = 0.020 lower half, 0.017 upper half) to a typical value (n = 0.013 upper and lower).
- Downstream improvements at the Back River WWTP increase the capacity of the plant and lower the HGL in the Outfall Interceptor. This is represented in the Outfall Sewershed model as a level boundary condition at the County Line that does exceed 48 feet. At 48 feet, the Outfall Interceptor and the Outfall Relief Sewer are approximately 90% full and the maximum HGL is 1 foot below the crown of the pipe.
- The tunnel is allowed to fill completely and after surcharging, the tunnel operates in a siphon mode.

A small dewatering pump (approximately 2 MGD capacity) would be used after an event to empty the tunnel. The dewatering pump is sized to empty the tunnel in 1 day.

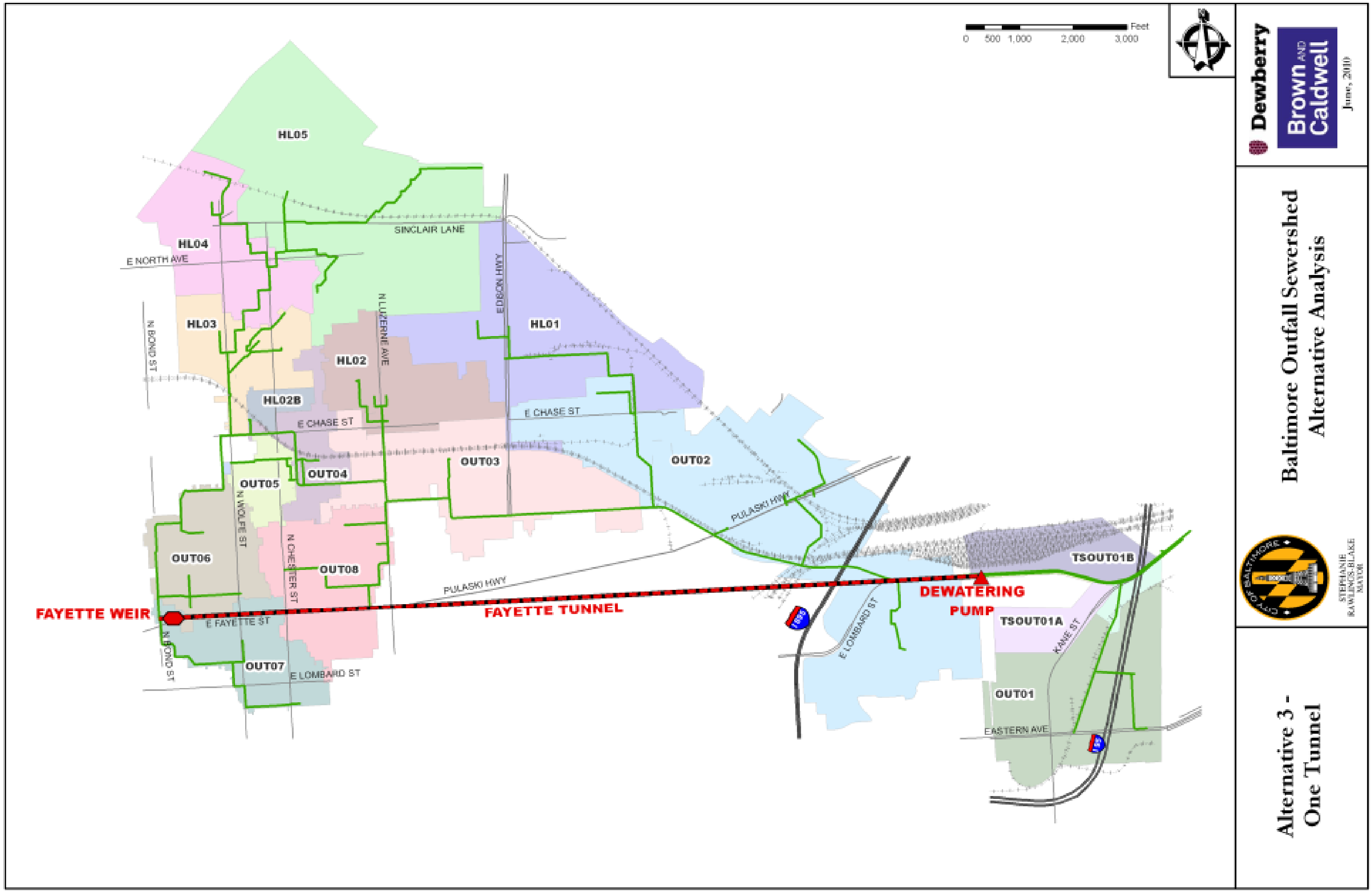


Figure 3.2 Alternative 3 Facilities: Storage/Conveyance Tunnel

Because Alternative 3 does not have a relief weir at Chase Street, this alternative is particularly sensitive to the conveyance capacity of the Outfall Interceptor and the 99-inch sewer. Cleaning sediment from the trunk sewers restores the conveyance capacity of the existing trunk sewers and reduces the degree of surcharging at the upstream end of the Outfall Interceptor in the vicinity of Chase Street and Durham Street.

A significant benefit of the tunnel alternative is that it provides an alternative, parallel flow path to the existing Outfall Interceptor. In the same way that the Outfall Relief sewer provides supplemental conveyance capacity (and in dry weather, a redundant flow path) to the Outfall Interceptor along Lombard St, a relief tunnel would provide an alternative, parallel flow path to the upstream section of the Outfall Interceptor. The upstream section of the Outfall Interceptor is a critical link in the overall conveyance system. The Outfall Interceptor is the only link to transport flow from the upstream sewersheds to the point where the Outfall Relief sewer starts to run parallel to it. A major incident that impairs the conveyance capacity of the existing Outfall Interceptor would have a large impact on the City. Major repairs and rehabilitation of the 100-year old Outfall Interceptor would be much easier to accommodate with a tunnel to serve as a redundant flow path. It would be possible to reverse the flow in the 99-inch sewer to redirect flow from the Outfall Interceptor to the Fayette tunnel if the Outfall Interceptor were to be closed for maintenance. Without a tunnel, large scale bypass pumping would be required to implement repairs to the Outfall Interceptor. Bypass pumping on this scale would be very expensive and disruptive.

3.4 Hydraulic Evaluation: Trunk Sewer Alternatives

Table 3.5 presents the required storage volumes at the Fayette and Chase weir locations to provided protection from overflows for the 2, 5, 10, 15, and 20-year return period design storms. Alternative 1 (two storage tanks) requires much greater storage volumes for any given return period than the other two alternatives because Alternative 1 does not assume any downstream improvements at the Back River WWTP.

Sediment removal is particularly helpful in all of the alternatives because more of the flow can be conveyed by the existing trunk sewers and less volume needs to be diverted at the Fayette weir. Without sediment in the Outfall Interceptor, the conveyance capacity is sufficient to pass all of the flow from the High Level Sewershed and much of the flow from the Low Level Sewershed. In the 2-year event (assuming a typical roughness value of n=0.013), no simulated overflows were experienced; therefore, Alternatives 2 and 3 are not needed until the 5-year event.

The simulation results are very sensitive to the Manning's roughness value used to calculate the conveyance capacity. With sediment in the large diameter trunk sewers, the calibration model used a Manning's roughness value of 0.020 for the lower half of the pipe and 0.017 for the upper half of the pipe. The calibration model values for roughness are very large; it is assumed that this high degree of roughness is due to the sediment in the sewers. After removal of the sediment, the roughness is assumed to have a value of 0.013 for the Manning's n parameter. The sensitivity to this assumption will be discussed later in the report.

Table 3.5
Trunk Sewer SSO Alternatives
Storage Volumes (MG)

Alternative	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
Alternative 1 Storage Tanks Sediment Removed but no downstream improvements	Fayette Weir Storage Tank	3.0	7.0	10.5	12.5	14.1
	Chase Weir Storage Tank	3.3	8.1	12.2	14.5	16.5
Alternative 2 Storage Tank Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Storage Tank	0	2.1	4.2	5.5	6.5
Alternative 3 Storage Tunnel Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Tunnel Siphon Mode	0	1.6	2.5	3.6	3.6

Table 3.6 presents the peak rates of excess flow into the relief facilities for the three alternatives and the various return period events. The peak flow rates over the weirs are useful for sizing the weir facilities.

Table 3.6
Trunk Sewer SSO Alternatives
Peak Rate of Excess Flow into Storage (MGD)

Alternative	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
Alternative 1 Storage Tanks Sediment Removed but no downstream improvements	Fayette Weir Storage Tank	61	69	93	95	97
	Chase Weir Storage Tank	42	65	84	88	95
Alternative 2 Storage Tank Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Storage Tank	0	37	66	70	73
Alternative 3 Storage Tunnel Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Tunnel Siphon Mode	0	42	71	75	75

Another alternative concept considered by the joint venture team was the use of a pump and force main instead of a storage tank or tunnel. A force main pump would need to be sized to accommodate the peak excess flow rates listed in Table 3.6. A force main alternative would require a pump a capacity on the order of 40 MGD for the 5-year event (and up to 80 MGD for the 20-year event). Assuming a peak velocity of 8 feet/second in a force main, the diameter of the force main would be approximately 36 inches. A force main of this size and length (approximately 17,000 LF) has a volume of 1 MG. This alternative was not further developed because of the high required pump capacity and large size of the required force main pipe. The significant storage volume of a force main pipe could be better used as a storage volume to attenuate the peak of the event. A tunnel can operate in siphon mode without the need for a high capacity force main pump.

Table 3.7 gives the peak flows at the County Line for the various alternatives. The peak flow in the table is the sum of the peak flows in the Outfall Interceptor and Outfall Relief Sewer. This peak flow is an indication of the treatment capacity that is require at the Back River WWTP. The details of the downstream improvements at the Back River WWTP are beyond the scope of this report, but this table indicates the magnitude of treatment capacity that is assumed to be available to make the Outfall Sewershed alternatives feasible.

Table 3.7
Trunk Sewer SSO Alternatives
Sum of Peak Flows At County Line (MGD)

Alternative	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
Alternative 1 Storage Tanks Sediment Removed but no downstream improvements	Outfall Interceptor + Outfall Relief	323	336	348	345	349
Alternative 2 Storage Tank Sediment Removed Downstream improvements at BR WWTP	Outfall Interceptor + Outfall Relief	394	416	431	438	445
Alternative 2 Storage Tunnel Sediment Removed Downstream improvements at BR WWTP	Outfall Interceptor + Outfall Relief	394	421	457	463	475

Possible dimensions of representative facilities are presented in Table 3.8. These sizes are given to help envision the general size of the proposed storage facilities. Alternatives 2 and 3 assume storage tanks at the weir sites that are 20 feet deep. The dimensions of the storage tanks are given in the table as the plan areas of the tanks in acres. Other tank configurations are possible; these dimensions are presented to describe the general size of the facilities that are needed. The property area needed for the tanks would be somewhat larger the nominal area of the tank itself.

Alternative 3 assumes a circular storage tunnel with a length of 17,000 LF. The Fayette tunnel would generally follow the alignment of Fayette Street from Bond Street to the Outfall Interceptor along Lombard Street near the connection from the Dundalk pump station (in the vicinity of 6000 E. Lombard Street, near Patterson High School).

No tunnel is required for Alternative 3 in the 2-year event provided that the assumptions of roughness and downstream improvements are achieved. A single 4-foot diameter tunnel is required for Alternative 3 in the 5-year event with sediment removed from the trunk sewers and assumed Manning's roughness value of 0.013. The tunnel size increases to a 6-foot diameter for the 20-year event.

Table 3.8
Trunk Sewer SSO Alternatives
Representative Dimension of Alternative Facilities

Alternative	Facility	2-yr	5-yr	10-yr	15-yr	20-yr
		Tank Plan Area (acres) Assuming 20 foot Tank Depth				
Alternative 1 Storage Tanks Sediment Removed but no downstream improvements	Fayette Weir Storage Tank	0.5	1.1	1.6	1.9	2.2
	Chase Weir Storage Tank	0.5	1.2	1.9	2.2	2.5
		Tank Plan Area (acres) Assuming 20 foot Tank Depth				
Alternative 2 Storage Tank Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Storage Tank	0	0.3	0.6	0.8	1.0
		Tunnel Diameter (feet) Siphon Mode Operation				
Alternative 3 Storage Tunnel Sediment Removed Downstream improvements at BR WWTP	Fayette Weir Tunnel Siphon Mode	0	4	5	6	6

3.5 Alternatives Evaluation based on Constructability Factors

3.5.1 Storage Tank Alternatives

The storage alternative considered for the elimination of the SSO along the Outfall Sewer and the 99-inch sewer requires the construction of storage tanks in the areas near Chase/Durham and Fayette/Bond Streets. The volume of the storage tanks required for the various wet weather events (2-year through 20-year) are documented in Table 3.5. The most significant factor regarding the constructability of these facilities is the availability of vacant land where the tanks could be located. As the two areas where the storage facilities would be located are fully developed, available property would be scarce. The most viable areas to consider would be public parks and old industrial sites that the city could purchase. Once the underground tanks and pump stations are constructed, the land could then be restored and used as green space, public parkland or recreational areas for sporting or playground facilities. The nominal area needed for the storage tank facilities for each alternative is listed in Table 3.8.

3.5.2 Conveyance Tunnels Alternatives

The tunnel alternatives considered for the elimination of the SSO along the Outfall Sewer and the 99-inch sewer require the construction a sewer along the east-west corridors from the Fayette/Bond intersection to the upstream end of the existing relief sewer. One significant factor regarding the constructability of these tunnels is the availability of land where the shafts could be located. As the corridor where the tunnel would be located is fully developed, available property is likely to be scarce. However, the space required for shafts, located about every 2,000 lf along each alignment, would be small – from ½ acre to 1 acre in size. The most viable areas to consider would be public parks, old industrial sites or parking lots that the city could lease for the construction period. Once the tunnel is constructed, the land could then be restored and used as green space, public parkland or recreational areas for sporting or playground facilities.

Since the excavation required for the tunnel would be deep, geotechnical issues could be a factor. The design of soft-ground tunnels would require a detailed geotechnical investigation program with the preparation of geotechnical data and baseline reports to mitigate the risks to construct these deep underground facilities. The areas targeted for the shafts have been previously disrupted, so environmental issues also need to be considered.

3.6 Alternatives Evaluation Based on Cost Factors

The costs developed for each alternative were used to assist in the selection of a recommended alternative to eliminate SSOs in the Outfall Sewershed. Because the 2-year event did not require any new facilities it was not used as the basis for this comparison of alternatives. Instead the cost comparison given in Table 3.9 is based on the 10-year event.

Cost values are estimated for the major facilities needed to eliminate SSOs from the trunk sewer; the costs for the smaller projects are not included. The unit construction cost values used for this table are contained in Appendix A. The total estimated cost values in Table 3.9 include the 42% allowance for contingencies and other project implementation tasks.

Table 3.9
Cost Comparison of 10-year Alternatives

Alternative	Alternative Facilities	Total Estimated Cost (Million $)
Alternative 1 Storage Tanks	Fayette Storage Tank, 10.5 MG Chase Storage Tank, 12.2 MG Sediment Removed	Fayette Tank: $90 Chase Tank: $104 Sediment Removal: $24 **Total: $218**
Alternative 2 Storage Tank	Fayette Storage Tank, 4.2 MG Sediment Removed Assuming Downstream Improvements at Back River WWTP	Fayette Tank: $36 Sediment Removal: $24 **Total: $60**
Alternative 3 Tunnel	Fayette Tunnel 5-ft Diameter x 17,000 LF Dewatering Pump, 2.5 MGD Sediment Removed Assuming Downstream Improvements at Back River WWTP	Fayette Tunnel: $112 Pump Station: $10 Sediment Removal: $24 **Total: $146**

Alternative 1 does not assume any downstream improvements at the Back River WWTP. This is the cost to manage the SSO problem with facilities in the Outfall Sewershed alone.

Alternatives 2 and 3 assume that there are downstream improvements at the Back River WWTP, but the cost of those downstream improvements are not accounted for in the table. The cost of Alternatives 2 and 3 are substantially lower than Alternative 1 because of the downstream improvements at the Back River WWTP. Even though the cost of Alternative 3 is greater than Alternative 2, the additional flexibility of the tunnel facilities merits consideration when choosing between the tank and tunnel concepts. The next section will provided further insight on the benefits of the tunnel and tank alternatives by evaluating the sensitivity of the results to the modeling assumptions.

3.7 Sensitivity of Simulation Results to Modeling Assumptions

3.7.1 Sensitivity to Manning's Roughness

The simulation results are very sensitive to the assumed roughness value once the sediment is removed from the large diameter trunk sewers. Manning's roughness values for concrete sewer pipes typically vary from 0.010 to 0.017. The value of 0.013 is commonly used as a design value to account for the roughness of the pipes including manholes and other sewer system features that result in additional roughness beyond that of a simple straight segment of uniform pipe.

Figure 3.3 shows the path of the pipe profile shown in Figure 3.4, which is the profile for the 2-year event with n = 0.013. The path of the hydraulic profile starts along the small branch sewer pipe near the Bethel Street overflow location (manhole S43E__016MH), continues downstream along the 99-inch sewer to the Eager Street overflow location (manhole S45CC_007MH), then along the Outfall Interceptor to the County Line.

Figure 3.4 shows the HGL along the length of the pipe, the pipe invert and crown, and the ground surface elevation. These figures are images from the InfoWorksCS® hydraulic modeling software.

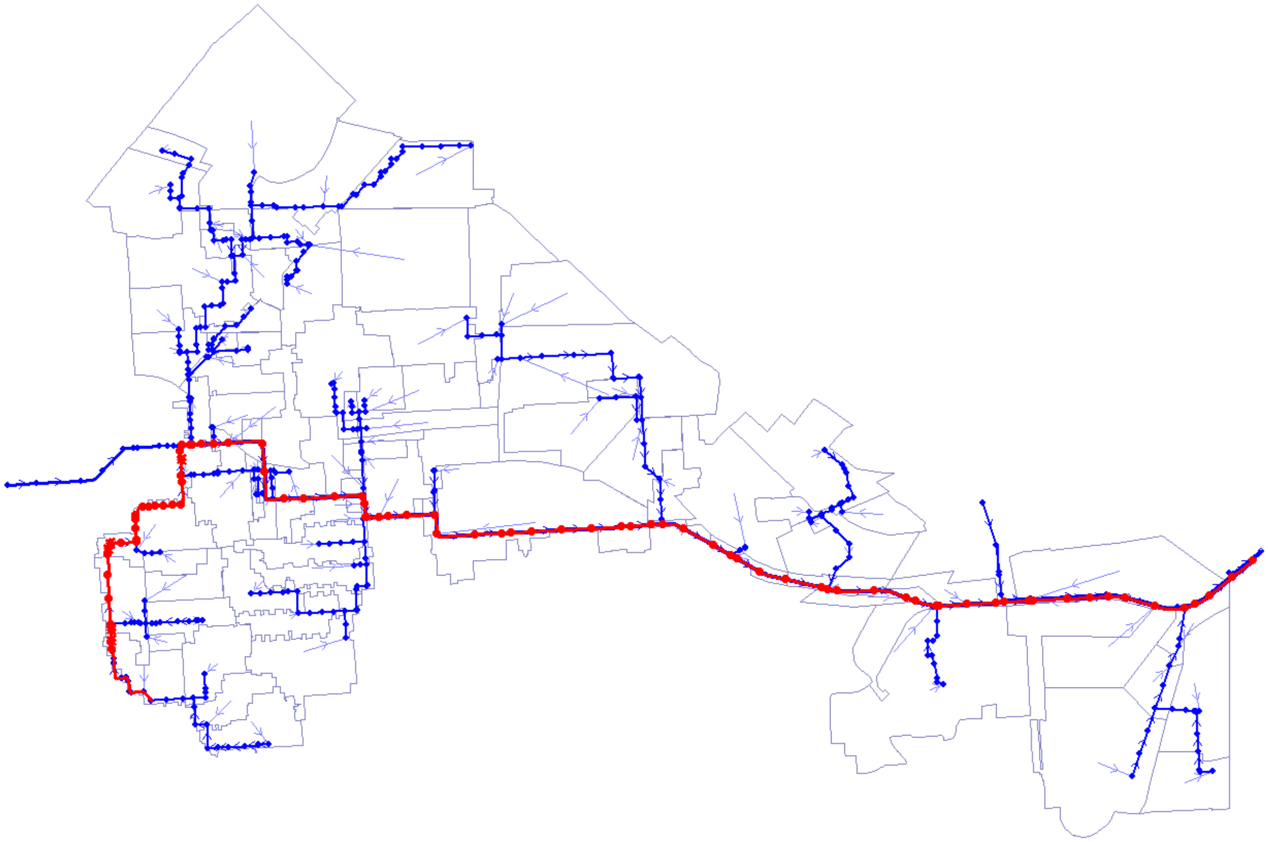


Figure 3.3 Path of Hydraulic Profile

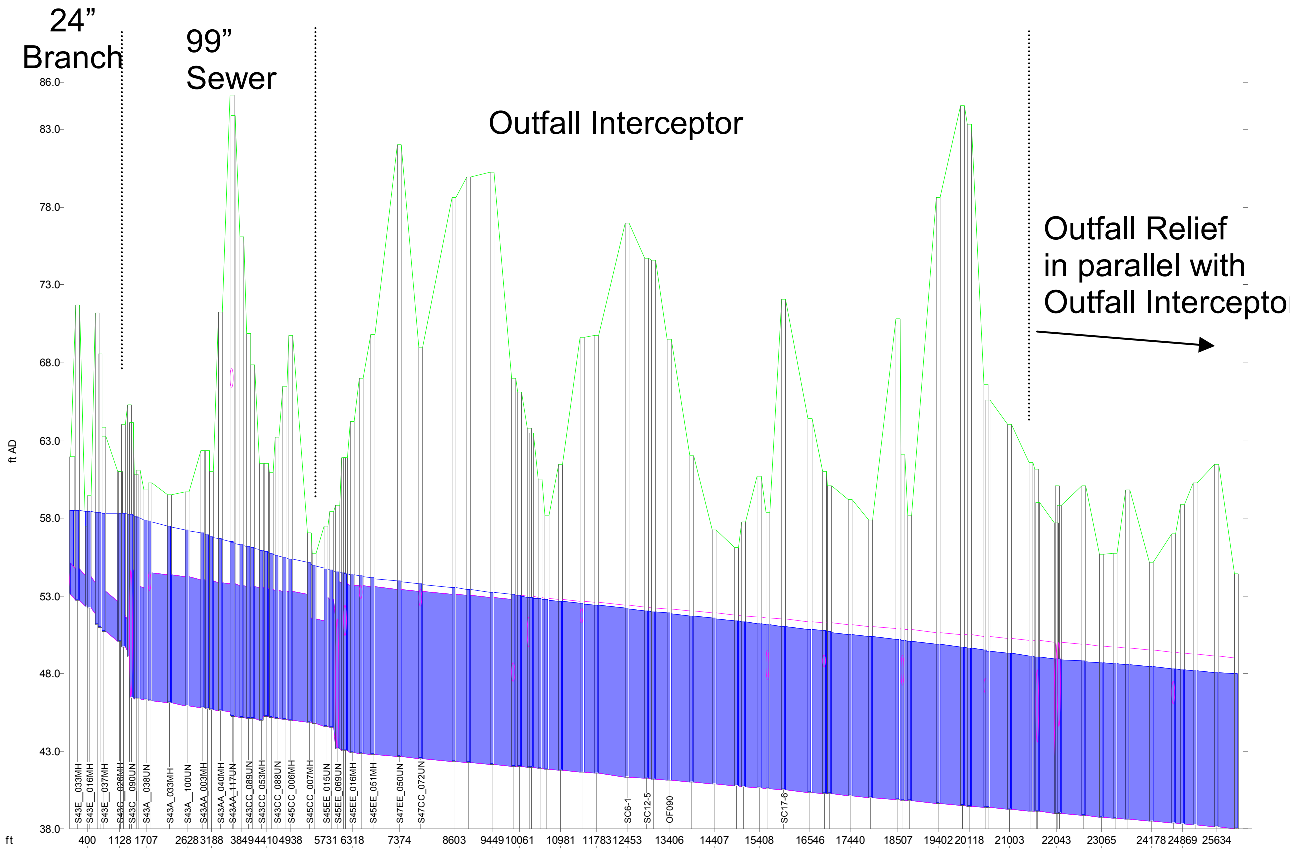


Figure 3.4 Hydraulic Profile for the 2-year Event: Assuming n = 0.013

There are no simulated overflows for the 2-year event when:

- sediment is removed
- roughness value is assumed to be a Manning’s n value of 0.013
- downstream improvements at the Back River WWTP allow the HGL at the County Line not to exceed 48 ft (90% full)

Even though there are no simulated overflows for this set of assumptions, the 99-inch sewer is surcharged. The freeboard at the Bethel Street location is 1 foot and the freeboard at the Eager Street location is 2 feet.

The actual roughness of the trunk sewers after removal of the sediment is unknown. If the model roughness value is increased slightly (n=0.014) a small simulated overflow volume (0.01 MG) results at Bethel Street and the freeboard at Eager Street is reduced to 0.5 feet.

If the model roughness value is further increased (n=0.015) small simulated overflow volumes result at Bethel Street (0.13 MG) and Eager Street (0.02 MG). Figure 3.5 shows the hydraulic profile for n=0.015. The adverse slope of the HGL in the small branch sewer is indicative of reverse flow in that pipe leading to the Bethel Street overflow. The steep slope of the HGL along the 99-inch sewer and along the Outfall Interceptor indicates that the flow exceeds the nominal full pipe capacity of the sewers. Near the downstream end of the profile, the HGL is below the crown of the pipe and the slope of the HGL is approximately equal to the pipes slope; this is because of the additional conveyance capacity provided by the Outfall Relief sewer along Lombard Street.

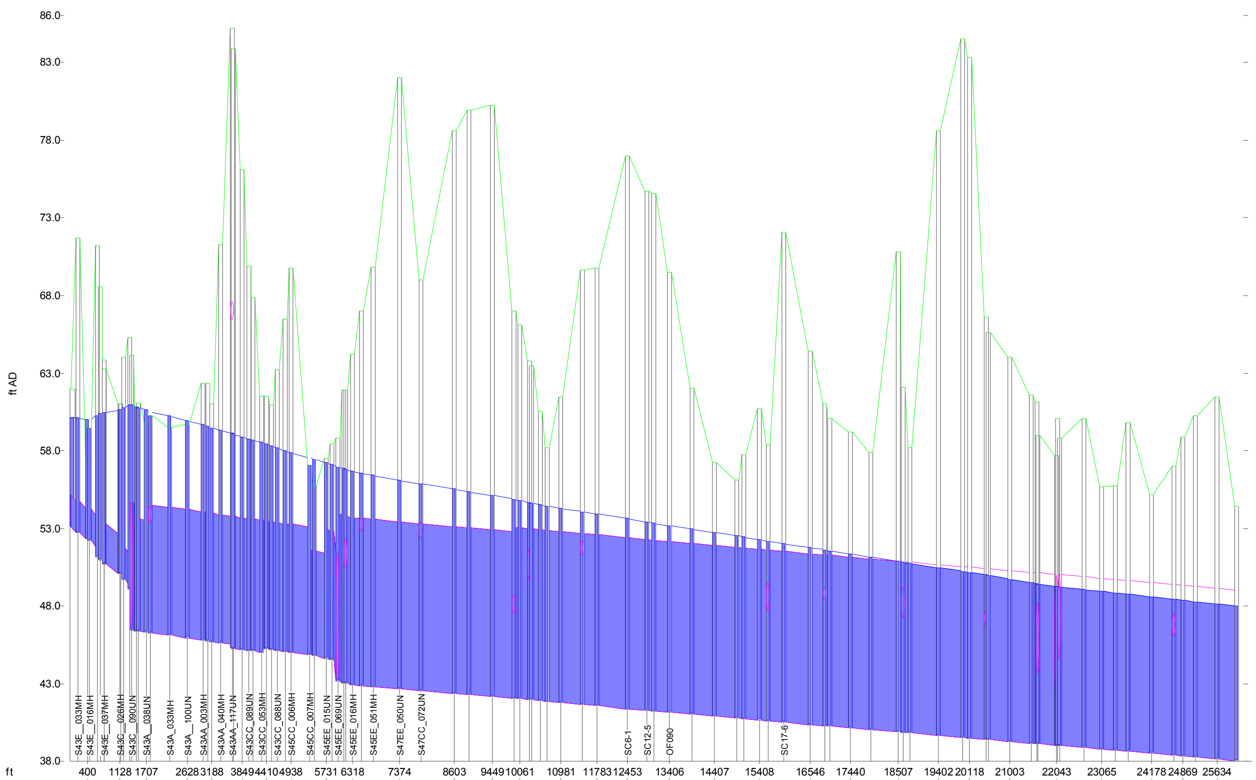


Figure 3.5 Hydraulic Profile for the 2-year Event: Assuming n = 0.015

3.7.2 Sensitivity to Eastern Avenue Pump Station Operations

The Eastern Avenue Pump Station has five pumping rates in the modeling boundary conditions used in the Outfall Sewershed model (as provided by the Technical Program Manager on 2009-09-18). Table 3.10 lists the pumping rates by the number of pumps online: the maximum rate for the 2-year event is 137 MGD and the maximum rate for the 10-year event is 160 MGD.

Table 3.10 Eastern Avenue Pump Station Capacity

Pump Online	Pump Discharge Rate (MGD)	Comment
1	38	Dry weather flow
2	76	
3	108	
4	137	Maximum rate for 2, 5-year events
5	160	Maximum rate for 10, 15, 20-year events

The full pipe capacity of the 99-inch sewer is approximately 130 MGD without sediment assuming n = 0.013. With a modest surcharge, the 99-inch sewer can convey the discharge of four pumps online (137 MGD) without overflowing when the Outfall Interceptor is not surcharged. All pumps online at the maximum pumping rate (160

MGD) exceeds the conveyance capacity of the 99-inch sewer and will lead to overflows at the Bethel Street location.

Guidelines in Section 7.6.2 of the BaSES Manual require that the design storms be evaluated for two scenarios, one with all pumps online, and another with the backup pumps offline. For the evaluation of the Outfall Sewershed model, the scenario with all pumps online produces the most severe condition.

Figure 3.6 shows the hydraulic profile for the 2-year event with all pumps online from the Eastern Avenue Pump Station. In this modified simulation of the 2-year event, there is a simulated overflow at Bethel St (0.03 MG) even though the pipe roughness is assumed to be n = 0.013 in the trunk sewers.

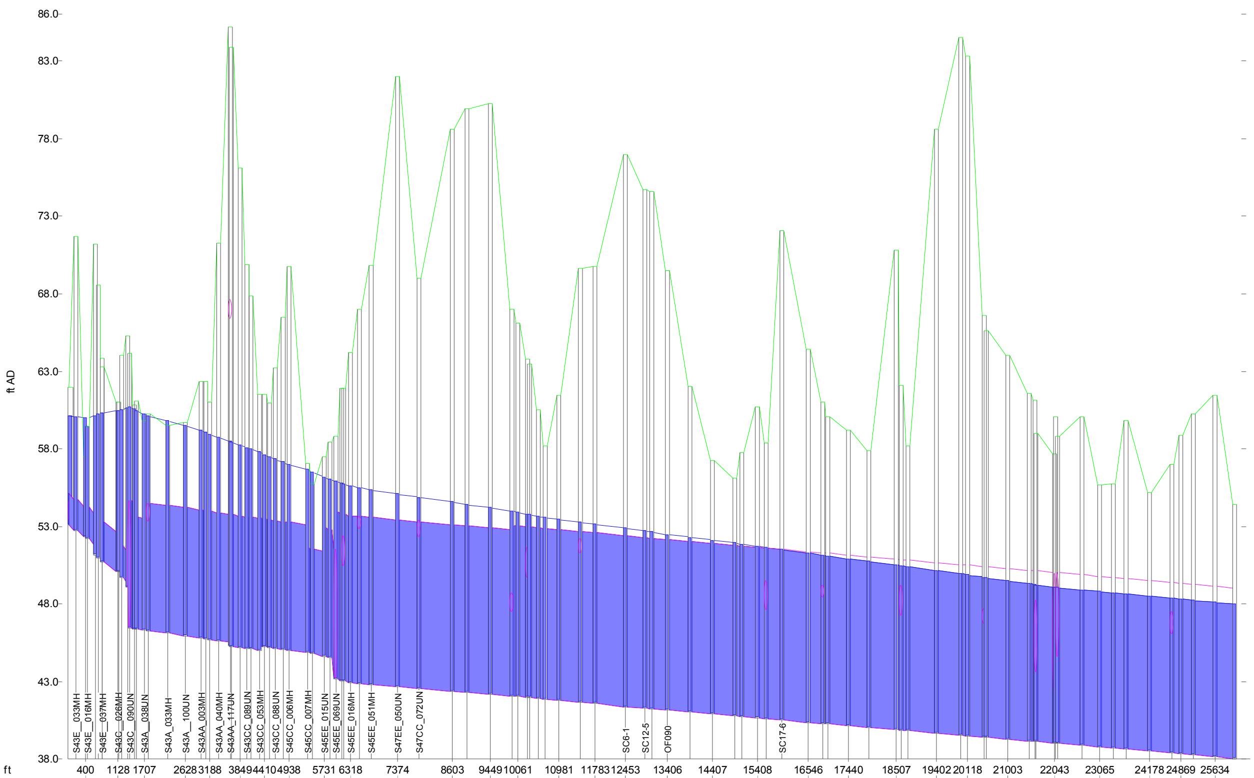


Figure 3.6 Hydraulic Profile for the 2-year Event
Assuming n = 0.013 and All Pumps Online (160 MGD) from the Eastern Avenue Pump Station

Under ideal modeling conditions, no alternatives facilities are required for the 2-year event but there is still a risk of an SSO because of the surcharging in the 99-inch sewer and the small amount of freeboard at Bethel Street. The risk of SSOs increases if the roughness is greater and the pumping rates are higher than the ideal conditions.

3.8 Alternative Facilities Evaluated for Sub-Optimal Conditions and Large Wet Weather Events

The 10-year alternative facilities presented in Table 3.6 above are either a 4.2 MG tank or a 5-foot diameter tunnel. The initial analysis results shown in Table 3.6 are for the nominal roughness conditions (n=0.013).

This section is a discussion of the performance of a 4.2 MG tank and a 5-foot tunnel for more extreme events and for a higher roughness assumption. Simulations using a 4.2 MG tank and a 5-foot tunnel were run for sub-optimal conditions and larger events to evaluate the robustness of each case.

The SSO volumes simulated during the Upstream Improvements evaluation provide the baseline for determining the volume of SSO removed. The baseline simulations do not assume any improvements downstream at the Back River WWTP and that sediment remains in the system (consequently the roughness remains at the calibration values, n=0.020 lower/0.017 upper).

All of the alternative simulations in this section assume “sub-optimal” conditions with roughness n=0.015 and all pumps online at the Eastern Avenue Pump Station. The alternatives also assume that there are downstream improvements at the Back River WWTP.

Figure 3.7 shows the simulated SSO volume for four cases:

- Upstream Improvements (baseline)
- Downstream Improvements and Sediment Removed (n=0.015)
- Alternative 2 (4.2 MG storage tank)
- Alternative 3 (5-foot diameter tunnel)

The improvements at the Back River WWTP make the single greatest reduction in SSO volume. Even under sub-optimal conditions, in the 2-year event, only 1% of the SSO volume remains due to the additional treatment capacity of the downstream improvements. In the 20-year event, only 10% of the baseline SSO volume remains.

Alternative 2 required a 4.2 MG tank for the 10-year event with nominal conditions. For sub-optimal conditions, the 4.2 MG tank eliminated simulated SSOs for the 2-year event and only 7% of the baseline SSO remains in the 20-year event.

Alternative 3 required a 5-foot diameter tunnel for the 10-year event with nominal conditions. For sub-optimal conditions, the 5-foot diameter tunnel eliminated the simulated SSOs for the 2-year event and only 2% of the baseline SSO remains for the 20-year event.

Both the tank and the tunnel provided significant protection for SSOs in the extreme events (15 and 10-year events), but the tunnel is more effective in minimizing overflows due to its ability to convey excess flow throughout the storm duration. A tunnel would also be more effective than a tank in back-to-back wet weather events because it does not rely on dewatering to restore the functionality of the facility.

These simulations also show that a facility sized for a 10-year event with nominal conditions is likely to provided protection against SSOs for a 2-year event in sub-optimal conditions.

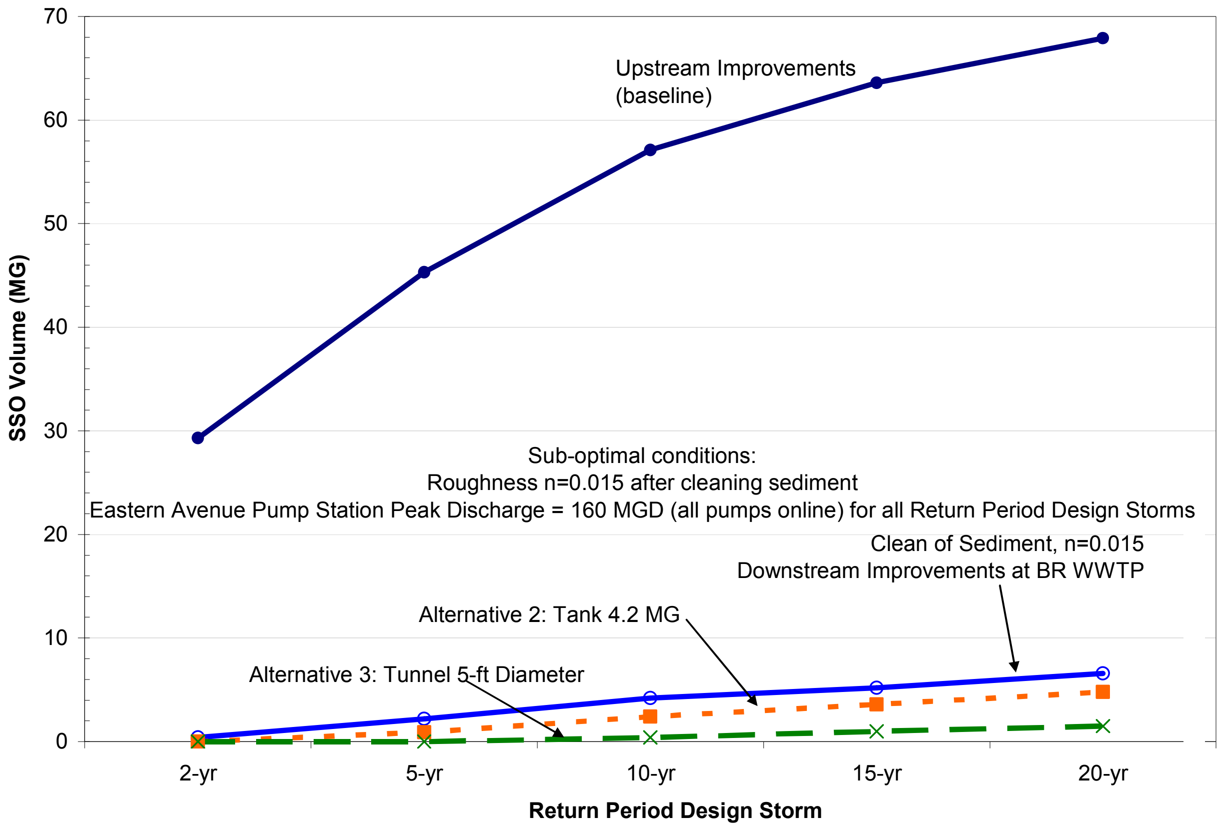


Figure 3.7 Simulated SSO Volume for Alternatives in Sub-Optimal Conditions

Figure 3.8 shows the sum of peak flows at the County Line for the Outfall Interceptor and the Outfall Relief sewer. In the Upstream Improvements (baseline) simulations, the sum of peak flows is less than 300 MGD. This rate is the approximate limit of flows at the County Line when there are no downstream improvements at the Back River WWTP.

The alternatives assume downstream improvements at the Back River WWTP so that greater flows and lower water levels are possible at the County Line. The alternative simulations assume additional treatment capacity is sufficient to allow the flow at the County Line to increase approximately 100 MGD more than the existing rate in the 2-year event. In the 20-year event the additional flow is approximately 140 MGD greater than the baseline flow for Alternative 2 and approximately 180 MGD greater for Alternative 3. The higher flows in Alternative 3 are the result of siphon flow through the tunnel.

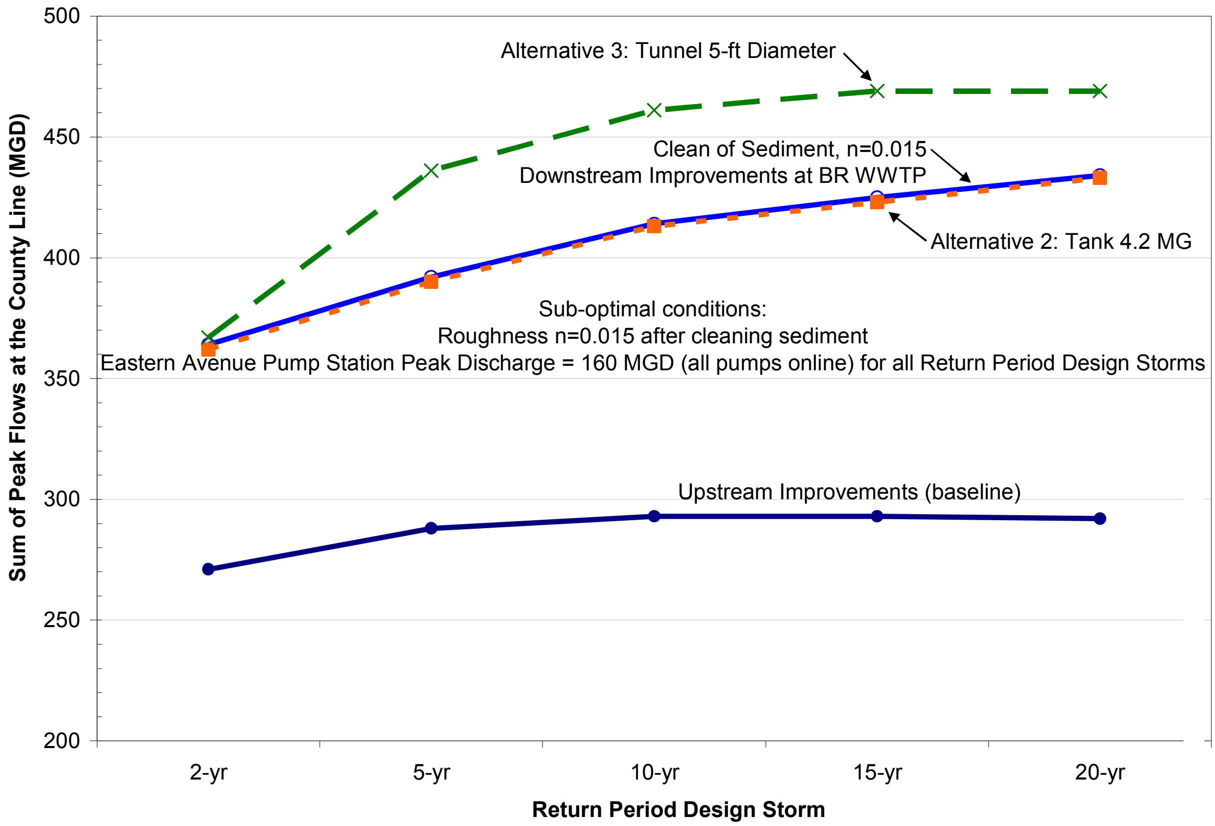


Figure 3.8 Sum of Peak Flows at the County Line for Alternatives in Sub-Optimal Conditions

4.0 Summary of Improvements

Downstream improvements at the Back River WWTP and the removal of sediment from the sewers to restore the conveyance capacities are the most effective changes to improve system performance and reduce the likelihood of overflows. No additional facilities are needed for the 2-year event in the Outfall Sewershed (if the assumed Manning's roughness value is accurate and the Eastern Avenue Pump Station does not operate at full capacity). Even for sub-optimum conditions, the downstream improvements and the removal of sediment are sufficient to remove 99% of the simulated SSO volume in the 2-year event compare to the baseline overflow volume.

A moderately sized storage tank or tunnel is needed at the Fayette relief point to fully eliminate SSOs for events greater than the 2-year storm and for sub-optimal conditions. Rather than defining a specific alternative recommendation, the findings of this evaluation and the summary cost tables below are presented for the purpose of discussion with the City. The cost of Alternative 2 (storage tank) is lower than the cost of Alternative 3 (tunnel). Therefore, Alternative 2 is the lowest cost approach to eliminating SSOs in the Outfall Sewershed.

Even though Alternative 3 (tunnel) is not the lowest cost option, it does provide greater flexibility and is more effective in reducing SSO volume for larger events. The advantages of a tunnel include:

- Relief for the 99-inch sewer when the Eastern Avenue Pump Station operates with all pumps on-line
- Effective reduction of SSO volume in extreme events (approximately 1 to 2% of baseline SSO volume remaining)
- Functional in back-to-back wet weather events because siphon mode operation does not require dewatering time like a storage tank
- Parallel/redundant flow path to the Outfall Interceptor (useful as a dry weather bypass if the Outfall Interceptor needs maintenance, cleaning, or repair).

The improvements needed for each of the design storms are summarized below for Alternative 3 (tunnel) for the nominal conditions. The tables presented in the summaries below itemize the recommended improvements and the costs to implement each improvement. The costs are given for 10 years (which is the span of potential implementation of the projects), from 2008 (the cost "base year") to 2017, escalated by 7% a year, as required by the methodology described in BaSES Manual, Section 8.3.2.1.

4.1 2-Year Improvements

Figure 4.1 presents the improvements recommended for the 2-year return period event; no additional facilities are required for the 2-year event assuming optimum conditions. Costs of the 2-year improvements are itemized in Table 4.1; the only cost in the Outfall Sewershed is the cost of removing the sediment. Not given in this report are the costs of the downstream improvements; specifically the cost of cleaning of the trunk sewers from the County Line to the Back River WWTP and the cost of capacity upgrades at the treatment plant.

Table 4.1
2-year Outfall Improvements
Alternative 3: Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1,600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29,000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3,600	tons	$1,800,000
					Subtotal	**$17,100,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$7,182,000
2008 Total Estimated Cost						**$24,282,000**
2009 Total Estimated Cost						**$25,982,000**
2010 Total Estimated Cost						**$27,801,000**
2011 Total Estimated Cost						**$29,747,000**
2012 Total Estimated Cost						**$31,829,000**
2013 Total Estimated Cost						**$34,057,000**
2014 Total Estimated Cost						**$36,441,000**
2015 Total Estimated Cost						**$38,992,000**
2016 Total Estimated Cost						**$41,721,000**
2017 Total Estimated Cost						**$44,641,000**

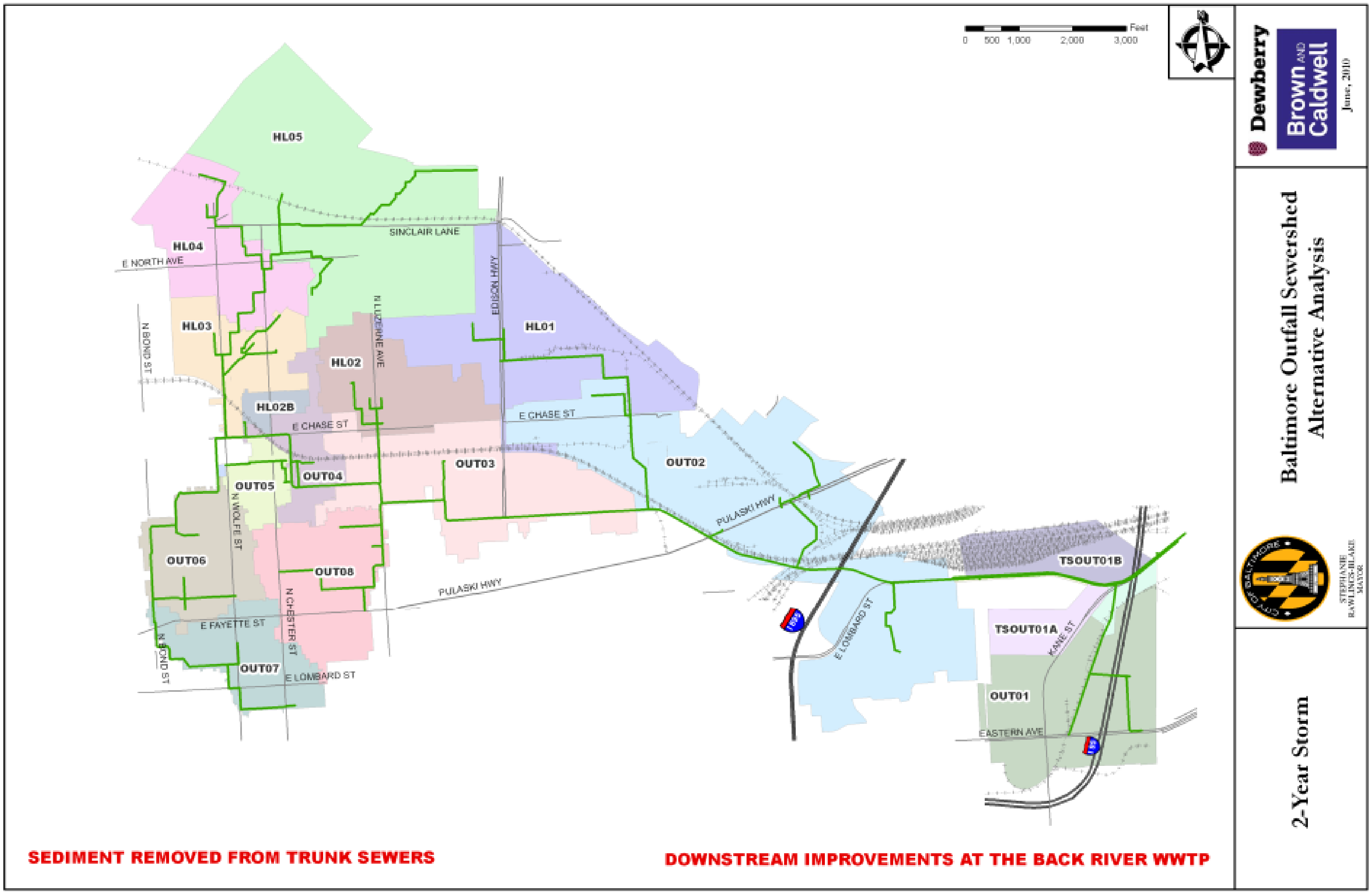


Figure 4.2 2-year Improvements for Alternative 3

4.2 5-Year Improvements

Figure 4.2 presents the improvements recommended for the 5-year return period event. A 4-foot diameter tunnel at the Fayette site is needed in the 5-year event along with sediment removal and downstream improvements at the Back River WWTP. The branch sewer in the HL04 meter basin area requires a small storage tank near Wolfe and Darley Streets. Costs of the 5-year improvements are itemized in Table 4.2.

Table 4.2
5-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
			Branch Sewer Improvements			
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.047	MG	$282,000
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
			Major Relief Facilities			
Fayette Tunnel	Fayette Storage Tunnel 4' x 17,000 LF	44.14	$/gal	1.6	MG	$70,533,060
	Dewatering Pump	3.00	$/gpd	2	MGD	$6,000,000
			Sediment Cleaning in Trunk Sewers			
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$95,008,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$39,903,000
		2008 Total Estimated Cost				**$134,911,000**
		2009 Total Estimated Cost				**$144,355,000**
		2010 Total Estimated Cost				**$154,460,000**
		2011 Total Estimated Cost				**$165,272,000**
		2012 Total Estimated Cost				**$176,841,000**
		2013 Total Estimated Cost				**$189,220,000**
		2014 Total Estimated Cost				**$202,465,000**
		2015 Total Estimated Cost				**$216,638,000**
		2016 Total Estimated Cost				**$231,803,000**
		2017 Total Estimated Cost				**$248,029,000**

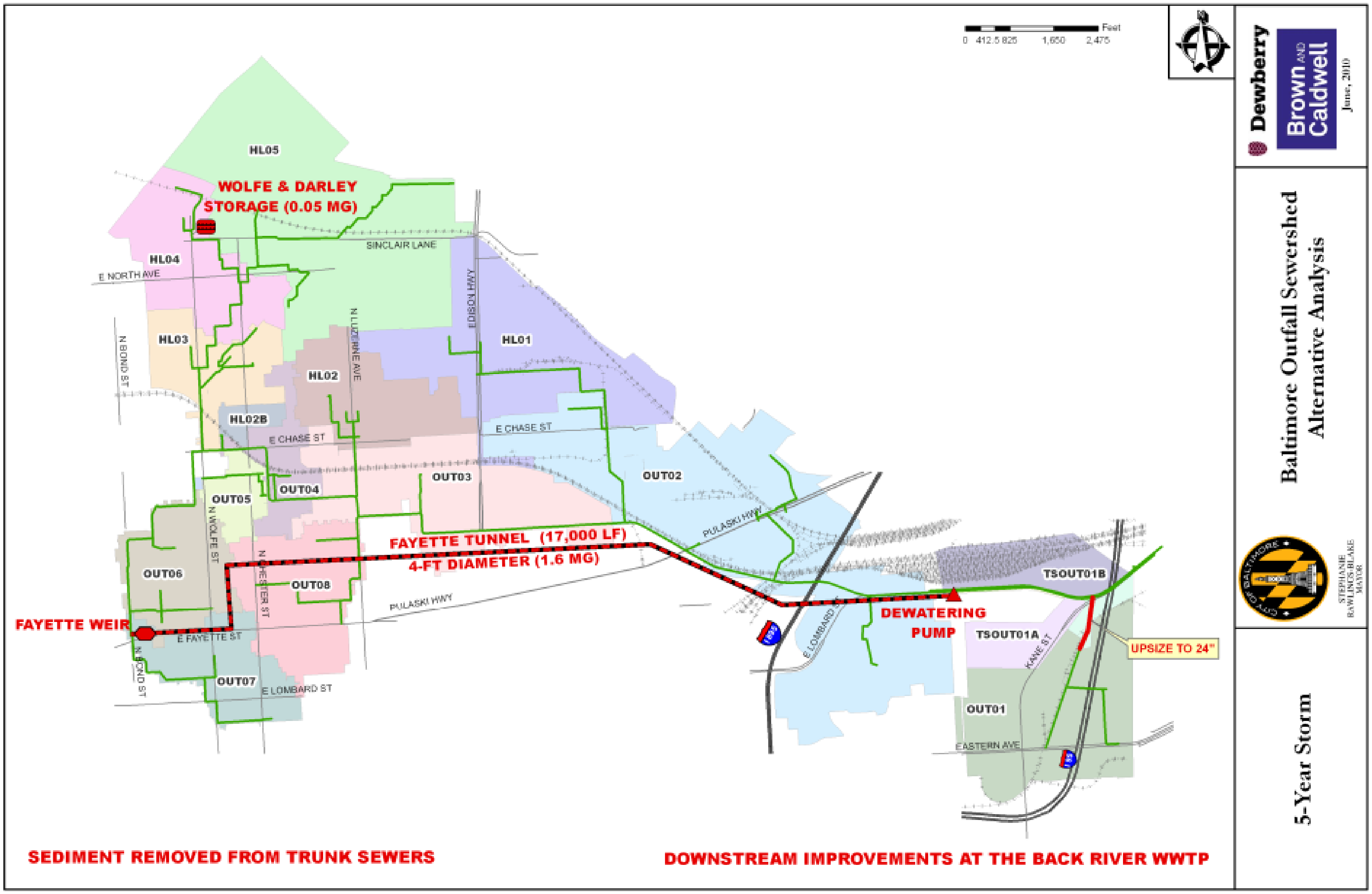


Figure 4.2 5-year Improvements for Alternative 3

4.3 10-Year Improvements

Figure 4.3 presents the improvements recommended for the 10-year return period event. For this level of protection, the Fayette Tunnel is further increased in size, and a few pipe replacement projects are recommended in meter basins HL05 and OUT01. This alternative assumes that sediment is cleaned from the trunk sewers and there are downstream improvements at the Back River WWTP. Costs of the 10-year improvements are itemized in Table 4.3.

Based on the results of the sensitivity analysis for sub-optimal conditions, the facilities needed for a 2-year level of protection in sub-optimal conditions are equivalent to those needed for the 10-year event with nominal conditions. Thus the costs presented in Table 4.3 are representative of the cost of facilities for a 2-year level of protection under sub-optimal conditions. These facilities are robust and provide protection with a greater degree of certainty. Even in extreme events greater than 10-year recurrence, these facilities are very effective in reducing the volume of SSOs, even if compete protection is not achieved.

4.4 15-Year Improvements

Figure 4.4 presents the improvements recommended for the 15-year return period event. New facilities added for the 15-year level of protection include a second small storage tank and a replacement sewer in the HL04 meter basin and an extension of the replacement sewer project in HL05. This alternative assumes that sediment is cleaned from the trunk sewers. Costs of the 15-year improvements are itemized in Table 4.4.

4.5 20-Year Improvements

Figure 4.5 presents the improvements recommended for the 20-year return period event. A couple of small replacement sewers in the HL02 meter basin are new for this event. The facilities needed for the 20-year event are very similar to those needed for the 15-year event.

This alternative assumes that sediment is cleaned from the trunk sewers and there are downstream improvements at the Back River WWTP. Costs of the 20-year improvements are itemized in Table 4.5.

Table 4.3
10-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.065	MG	$390,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
OUT01 Upper Section	18" Replacement Pipe	585	$/LF	400	LF	$234,000
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 5' x 17,000 LF	31.65	$/gal	2.5	MG	$79,023,110
	Dewatering Pump	2.84	$/gpd	2.5	MGD	$7,100,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$105,286,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$44,220,000
2008 Total Estimated Cost						**$149,506,000**
2009 Total Estimated Cost						**$159,971,000**
2010 Total Estimated Cost						**$171,169,000**
2011 Total Estimated Cost						**$183,151,000**
2012 Total Estimated Cost						**$195,972,000**
2013 Total Estimated Cost						**$209,690,000**
2014 Total Estimated Cost						**$224,368,000**
2015 Total Estimated Cost						**$240,074,000**
2016 Total Estimated Cost						**$256,879,000**
2017 Total Estimated Cost						**$274,861,000**

Table 4.4 15-year Outfall Improvements Alternative 3: Tunnel, Sediment Removed						
Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL04 Wolfe St	12" Replacement Pipe	495	$/LF	554	LF	$274,130
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.058	MG	$348,000
HL04 North&Chester Storage	Storage Tank	6	$/gal	0.073	MG	$438,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
HL05 Sinclair Lane	15" Replacement Pipe	585	$/LF	751	LF	$439,340
OUT01 Upper Section	21" Replacement Pipe	1080	$/LF	1599	LF	$1,726,920
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 6' x 17,000 LF	23.37	$/gal	3.6	MG	$84,023,660
	Dewatering Pump	2.53	$/gpd	4.00	MGD	$10,120,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$115,909,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$48,682,000
2008 Total Estimated Cost						**$164,591,000**
2009 Total Estimated Cost						**$176,112,000**
2010 Total Estimated Cost						**$188,440,000**
2011 Total Estimated Cost						**$201,631,000**
2012 Total Estimated Cost						**$215,745,000**
2013 Total Estimated Cost						**$230,847,000**
2014 Total Estimated Cost						**$247,006,000**
2015 Total Estimated Cost						**$264,296,000**
2016 Total Estimated Cost						**$282,797,000**
2017 Total Estimated Cost						**$302,593,000**

Table 4.5
20-year Outfall Improvements
Alternative 3: Tunnel, Sediment Removed

Site	Improvement	Unit Cost		Quantity		Cost
Branch Sewer Improvements						
HL02 Milton Ave	15" Replacement Pipe	585	$/LF	46	LF	$26,910
HL02 Luzerne St	24" Replacement Pipe	1080	$/LF	271	LF	$292,680
HL04 Wolfe St	12" Replacement Pipe	495	$/LF	554	LF	$274,130
HL04 Wolfe&Darley Storage	Storage Tank	6	$/gal	0.074	MG	$444,000
HL04 North&Chester Storage	Storage Tank	6	$/gal	0.107	MG	$642,000
HL05 Collington Ave	15" Replacement Pipe	585	$/LF	592	LF	$346,320
HL05 Sinclair Lane	15" Replacement Pipe	585	$/LF	751	LF	$439,340
OUT01 Upper Section	21" Replacement Pipe	1080	$/LF	1599	LF	$1,726,920
OUT01 Lower Section	24" Replacement Pipe	1080	$/LF	1012	LF	$1,092,960
Major Relief Facilities						
Fayette Tunnel	Fayette Storage Tunnel 6' x 17,000 LF	23.37	$/gal	3.6	MG	$84,023,660
	Dewatering Pump	2.53	$/gpd	4.00	MGD	$10,120,000
Sediment Cleaning in Trunk Sewers						
99-inch Sewer	Sediment Cleaning	500	$/ton	1600	tons	$800,000
Outfall Interceptor	Sediment Cleaning	500	$/ton	29000	tons	$14,500,000
Outfall Relief Sewer	Sediment Cleaning	500	$/ton	3600	tons	$1,800,000
					Subtotal	**$116,529,000**
Engineering. Design, Construction Management/Inspection, Administration, Post-Engineering Services, Contingency (42%)						$48,942,000
2008 Total Estimated Cost						**$165,471,000**
2009 Total Estimated Cost						**$177,054,000**
2010 Total Estimated Cost						**$189,448,000**
2011 Total Estimated Cost						**$202,709,000**
2012 Total Estimated Cost						**$216,899,000**
2013 Total Estimated Cost						**$232,082,000**
2014 Total Estimated Cost						**$248,328,000**
2015 Total Estimated Cost						**$265,711,000**
2016 Total Estimated Cost						**$284,311,000**
2017 Total Estimated Cost						**$304,213,000**

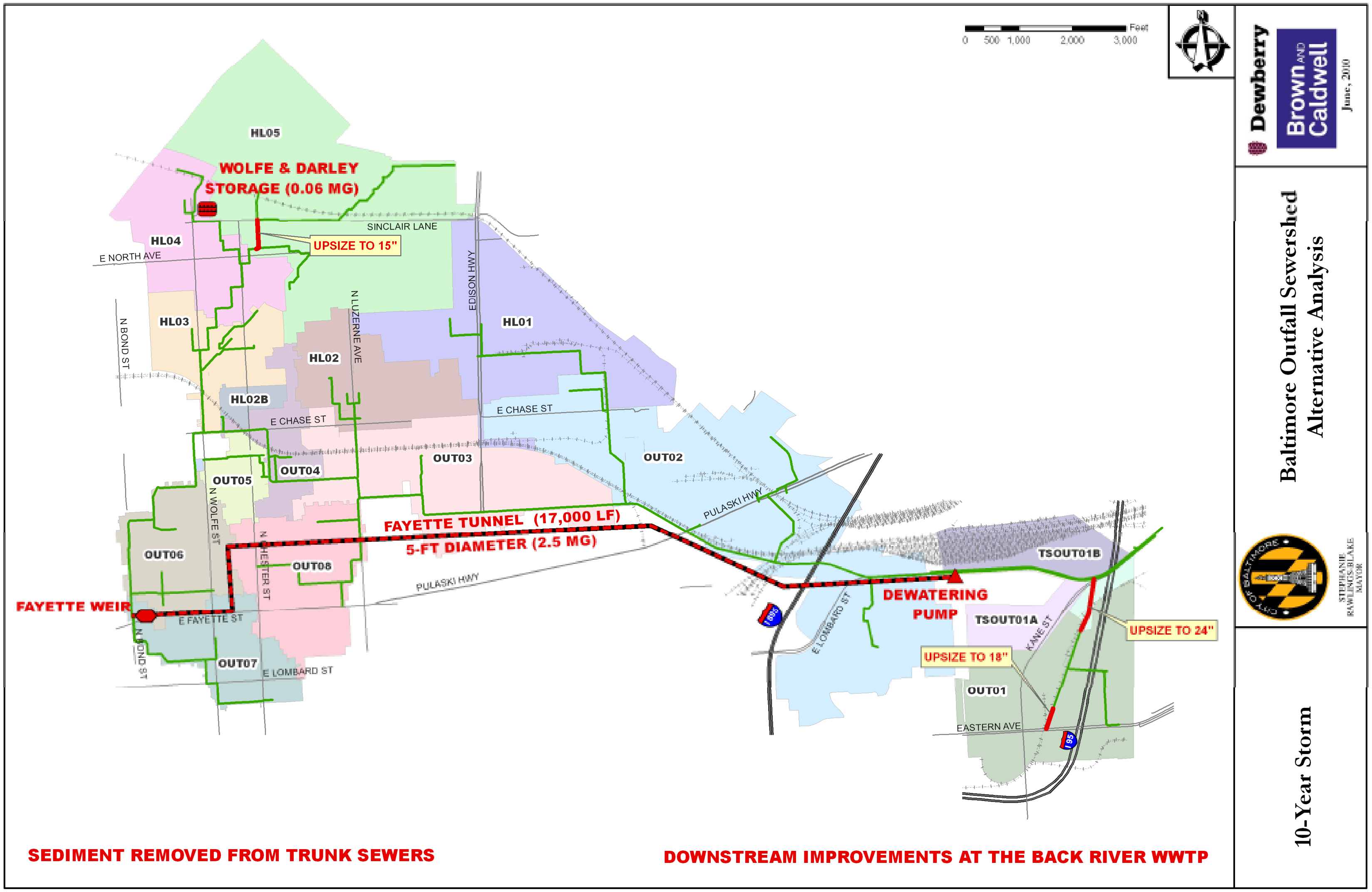


Figure 4.3 10-year Improvements for Alternative 3

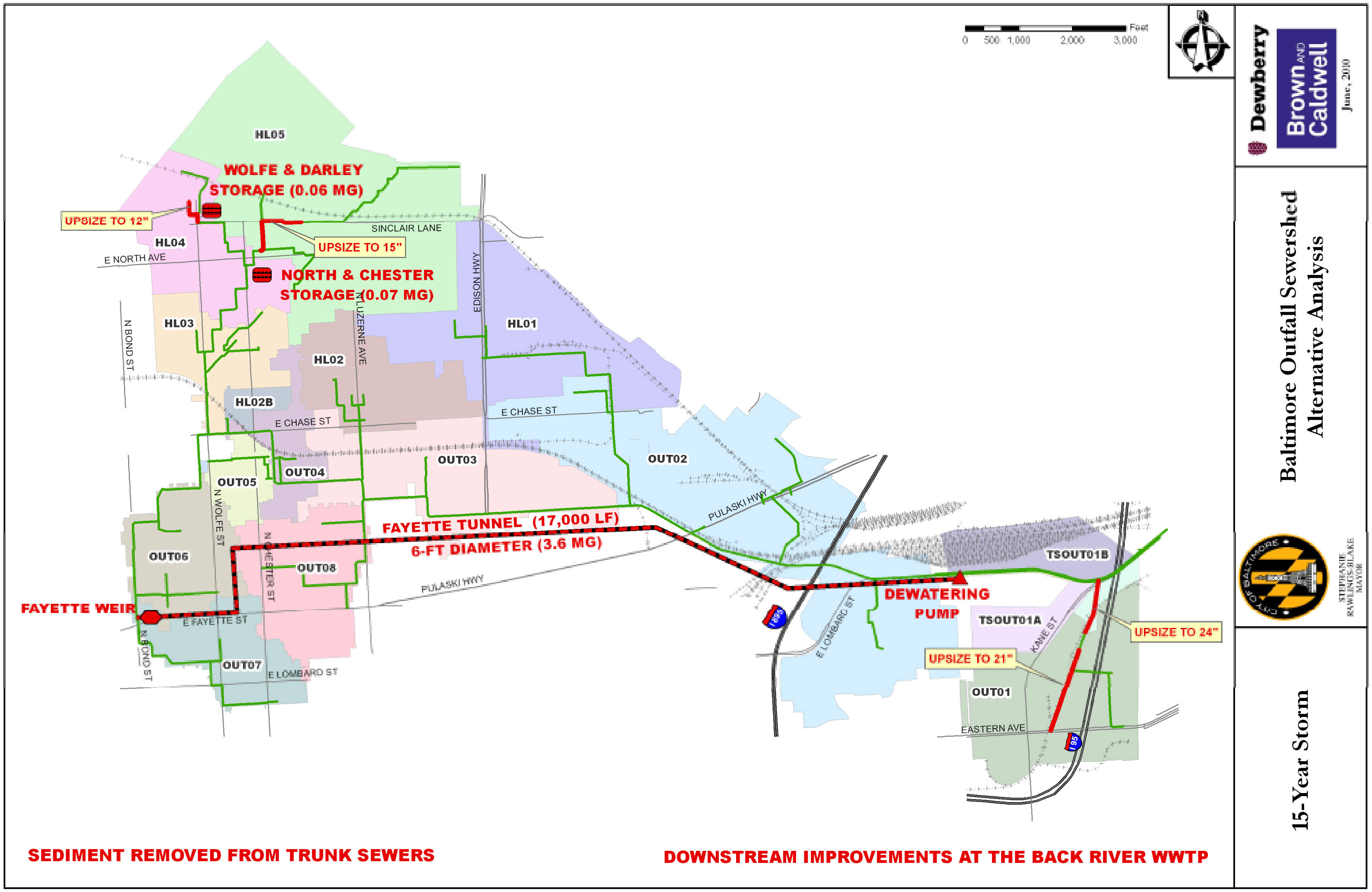


Figure 4.4 15-year Improvements for Alternative 3

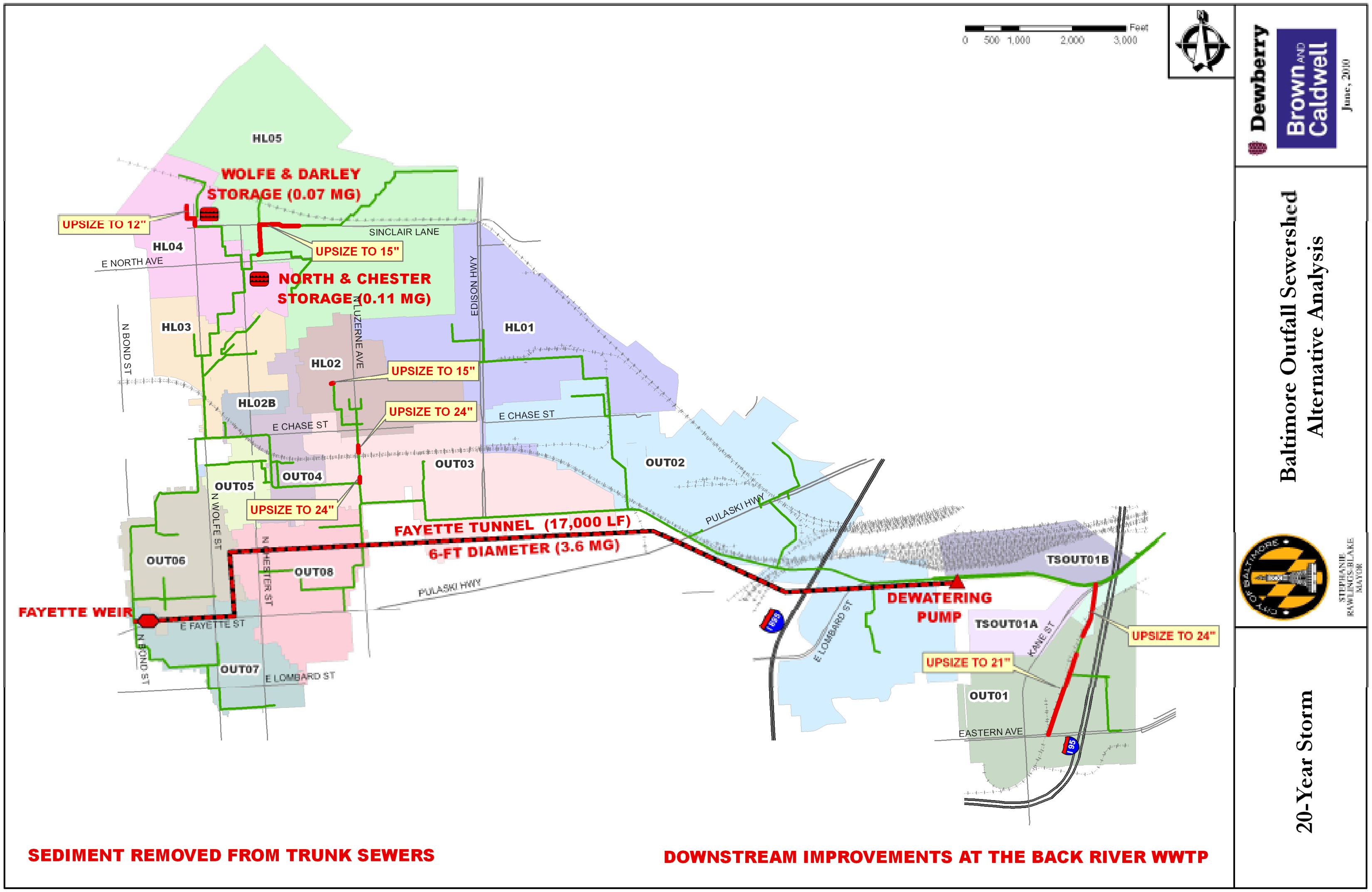


Figure 4.5 20-year Improvements for Alternative 3

4.6 Summary of Costs

Figure 4.6 shows the total costs for Alternatives 1, 2, and 3. Alternative 1 does not assume any downstream improvements at the Back River WWTP. This is the cost to manage the SSO problem within the Outfall sewershed with facilities located in the Outfall Sewershed alone. Alternative 1 does not address peak flows into the Back River WWTP that exceed the plant's existing treatment capacity.

Alternatives 2 and 3 assume that there are downstream improvements at the Back River WWTP, but the cost of those downstream improvements are not accounted for in this cost summary. The cost of Alternatives 2 and 3 are substantially lower than Alternative 1 because of the downstream improvements at the Back River WWTP. Even though the cost of Alternative 3 is greater than Alternative 2, the additional flexibility of the tunnel facilities merits consideration when choosing between the tank and tunnel concepts.

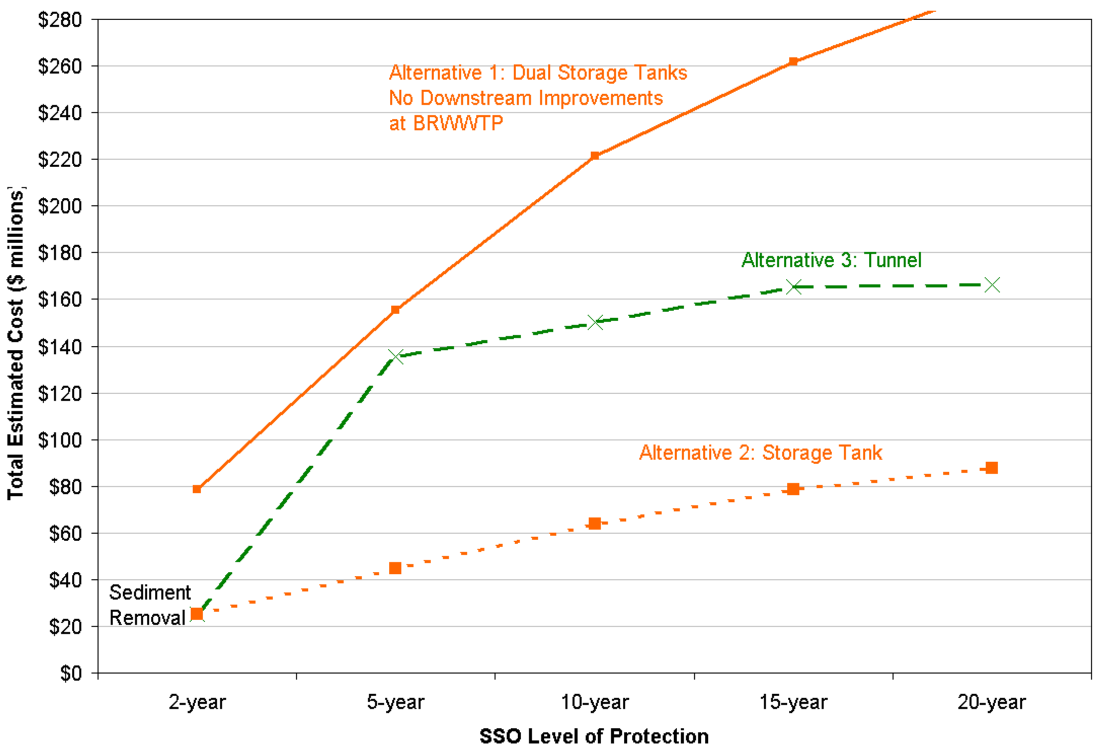


Figure 4.6 2008 Total Estimated Cost of Alternative 3

Construction costs were developed for all alternatives evaluated. To develop the estimated costs of construction, standard unit costs for sewer point repairs, sewer lining, sewer replacement, sewer cleaning, and manhole rehabilitation/replacement were provided by the City in 2008 dollars. The construction costs provided were fully loaded costs to address such items as mobilization, maintenance of traffic, paving restoration, bypass pumping and miscellaneous (non-sanitary) utility work. For costs not provided by the City (large diameter tunnels and pumping stations) recent projects within the

City and surrounding areas were reviewed to assist in estimating the most probable fully loaded cost of construction.

In addition to these construction costs, an additional 42 percent was added to accommodate engineering design, construction management/inspection, administration, post-award engineering services and contingencies. A 7 percent annual inflation rate is used to project costs for years beyond 2008.

Alternative 3 total estimated costs for the Outfall Sewershed improvements are summarized in Table 4.6 for the 2, 5, 10, 15, and 20-year events; the costs are inflated 7% per year for the recommended projects depending upon the year they might be implemented (from 2008 through 2017). The total estimated costs are under the column heading "Cumulative" in Table 4.6 for the 5, 10, 15, and 20-year events. The "Additional" cost column in the table is the incremental cost of facilities from one design storm level of protection to the next.

Table 4.7 is a summary of total estimated cost normalized by the volume of SSO removed. The units are dollars per gallon of SSO removed. The cumulative cost divided by the cumulative SSO volume removed is a direct normalization of the total cost by the total SSO volume. For example: The 2-year facilities removed 29.3 MG of SSO at a cost of $24 million; thus the unit cost is $0.83 per gallon of SSO removed. The 2-year facilities eliminate all of the SSOs in the 2-year event.

Incremental normalized cost values are also given in the table under the "Additional" columns. The additional costs per additional gallon of SSO volume removed were developed in the following manner: The 2-year facilities are effective in removing much of the SSO volume for the 5-year event, but the remaining SSO volume is 0.32 MG with the 2-year facilities in place. The additional cost of the 5-year facilities is $111 million compared to the 2-year facilities. The 5-year facilities are needed to remove the 0.32 MG of SSO that would remain if the 2-year facilities were in place. Therefore, the normalized additional cost is $346 per gallon of additional SSO removed.

The step wise progression was used to determine the additional SSO that could be removed by the 10-year facilities compared to the SSO remaining with the 5-year facilities. The normalized additional cost is $730 per gallon of additional SSO removed to reach the 10-year level of protection.

Likewise, the analysis determined the additional costs and the additional SSO volumes removed by the 15 and 20-year facilities. The additional volumes removed in these cases are negligible; therefore, the normalized additional costs are undefined.

The additional SSO removed is a relatively small volume because facilities sized for a smaller event are very effective at removing most of the SSO volume in a larger event, even though they may not be adequate to remove 100% of the SSO volume. As a result, the normalized costs ($/gallon) to remove the additional SSO volumes are extremely high.

Table 4.6 Total Estimated Outfall Improvement Costs										
Projected Year	**2-yr Cost**	**5-yr**		**10-yr**		**15-yr**		**20-yr**	
		Additional	Cumulative	Additional	Cumulative	Additional	Cumulative	Additional	Cumulative
2008	$24,282,000	$110,629,000	$134,911,000	$14,595,000	$149,506,000	$15,085,000	$164,591,000	$880,000	$165,471,000
2009	$25,982,000	$118,373,000	$144,355,000	$15,616,000	$159,971,000	$16,141,000	$176,112,000	$942,000	$177,054,000
2010	$27,801,000	$126,659,000	$154,460,000	$16,709,000	$171,169,000	$17,271,000	$188,440,000	$1,008,000	$189,448,000
2011	$29,747,000	$135,525,000	$165,272,000	$17,879,000	$183,151,000	$18,480,000	$201,631,000	$1,078,000	$202,709,000
2012	$31,829,000	$145,012,000	$176,841,000	$19,131,000	$195,972,000	$19,773,000	$215,745,000	$1,154,000	$216,899,000
2013	$34,057,000	$155,163,000	$189,220,000	$20,470,000	$209,690,000	$21,157,000	$230,847,000	$1,235,000	$232,082,000
2014	$36,441,000	$166,024,000	$202,465,000	$21,903,000	$224,368,000	$22,638,000	$247,006,000	$1,322,000	$248,328,000
2015	$38,992,000	$177,646,000	$216,638,000	$23,436,000	$240,074,000	$24,222,000	$264,296,000	$1,415,000	$265,711,000
2016	$41,721,000	$190,082,000	$231,803,000	$25,076,000	$256,879,000	$25,918,000	$282,797,000	$1,514,000	$284,311,000
2017	$44,641,000	$203,388,000	$248,029,000	$26,832,000	$274,861,000	$27,732,000	$302,593,000	$1,620,000	$304,213,000

Table 4.7
Total Estimated Outfall Improvement Costs per Gallon SSO Removed

SSO Volume (MG)	**Upstream Improvements 2-yr**	**5-yr** Remaining with 2-yr Facilities	**5-yr** Upstream Improvements	**10-yr** Remaining with 5-yr Facilities	**10-yr** Upstream Improvements	**15-yr** Remaining with 10-yr Facilities	**15-yr** Upstream Improvements	**20-yr** Remaining with 15-yr Facilities	**20-yr** Upstream Improvements
	29.3	0.32	**45.3**	0.02	**57.1**	negligible	**63.6**	negligible	**67.9**

SSO Volume Removed (MG)	**2-yr**	**5-yr** Additional SSO Removed by 5-yr Facilities	**5-yr** Cumulative SSO Removed	**10-yr** Additional SSO Removed by 10-yr Facilities	**10-yr** Cumulative SSO Removed	**15-yr** Additional SSO Removed by 15-yr Facilities	**15-yr** Cumulative SSO Removed	**20-yr** Additional SSO Removed by 20-yr Facilities	**20-yr** Cumulative SSO Removed
	29.3	0.32	**45.3**	0.02	**57.1**	negligible	**63.6**	negligible	**67.9**

Projected Year	**2-yr Cost**	**5-yr** Additional	**5-yr** Cumulative	**10-yr** Additional	**10-yr** Cumulative	**15-yr** Additional	**15-yr** Cumulative	**20-yr** Additional	**20-yr** Cumulative
2008	$0.83	$346.00	$2.98	$730.00	$2.62	undefined	$2.59	undefined	$2.44
2009	$0.89	$370.00	$3.19	$781.00	$2.80	undefined	$2.77	undefined	$2.61
2010	$0.95	$396.00	$3.41	$835.00	$3.00	undefined	$2.96	undefined	$2.79
2011	$1.02	$424.00	$3.65	$894.00	$3.21	undefined	$3.17	undefined	$2.99
2012	$1.09	$453.00	$3.90	$957.00	$3.43	undefined	$3.39	undefined	$3.19
2013	$1.16	$485.00	$4.18	$1,024.00	$3.67	undefined	$3.63	undefined	$3.42
2014	$1.24	$519.00	$4.47	$1,095.00	$3.93	undefined	$3.88	undefined	$3.66
2015	$1.33	$555.00	$4.78	$1,172.00	$4.20	undefined	$4.16	undefined	$3.91
2016	$1.42	$594.00	$5.12	$1,254.00	$4.50	undefined	$4.45	undefined	$4.19
2017	$1.52	$636.00	$5.48	$1,342.00	$4.81	undefined	$4.76	undefined	$4.48

APPENDIX A: COSTS

The cost for small diameter sewer replacement and storage tanks (Sections 1 and 2 below) are from the BaSES Manual, Section 8.3.2.1 (Table 8-34). The costs for construction of large diameter soft-ground tunnels and dewatering pump stations were not included in the BaSES Manual. Therefore, the required costs were prepared by an independent cost estimating effort performed during the preparation of this report.

1. Sewer Replacement Costs Derived from BaSES Manual

Diameter	Cost per LF	Loaded Cost per LF	(Open Cut Construction)
8”	$ 150	$ 270	
12”	$ 275	$ 495	
18”	$ 325	$ 585	
24”	$ 600	$1,080	
30”	$ 800	$1,440	
36”	$ 850	$1,530	
42”	$ 900	$1,620	
48”	$1,000	$1,710	
54”	$1,000	$1.800	
60”	$1,050	$1,890	

2. Storage Tank Costs

The unit cost of $6/gallon of storage, used to determine construction costs for storage tanks in this report, is provided by the City of Baltimore in the BaSES Manual. That cost includes the cost of the pumps needed to dewater the tanks.

3. Conveyance/Storage Tunnel Costs

The unit costs per linear foot to construct various sized conveyance/storage tunnels are based on a 17,000 lf of soft-ground tunnel, about 40 to 80 feet deep. It is assumed that the cost for shafts and ancillary facilities are included in the unit cost for each size tunnel, excluding pump station costs. Pump station costs were developed separately, and are presented in Section 4 below.

Unit Cost/lf of Tunnel	Storage Volume	Cost per Gallon of Storage
4 foot diameter = $4,154/lf	1.6 MG	$44.14 per gallon
5 foot diameter = $4,654/lf	2.5 MG	$31.65 per gallon
6 foot diameter = $4,949/lf	3.6 MG	$23.37 per gallon
8 foot diameter = $5,541/lf	6.4 MG	$14.72 per gallon
9 foot diameter = $5,894/lf	8.1 MG	$12.37 per gallon
10 foot diameter = $6,111/lf	10.0 MG	$10.39 per gallon
12 foot diameter = $6,702/lf	14.4 MG	$ 7.91 per gallon

14 foot diameter = $7,317/lf	19.6 MG	$ 6.35 per gallon
16 foot diameter = $7,903/lf	25.6 MG	$ 5.25 per gallon
18 foot diameter = $8,461/lf	32.4 MG	$ 4.44 per gallon
20 foot diameter = $9,035/lf	40.0 MG	$ 3.85 per gallon
22 foot diameter = $9,596/lf	48.4 MG	$ 3.37 per gallon

4. Tunnel Dewatering Pump Station Cost

The costs for tunnel dewatering pump stations are listed below in the sizes that correspond to the volume of the various sized tunnels. In this report, the dewatering pump stations are sized so that the tunnels can be evacuated within one day.

Cost of Dewatering Pumps	Cost per Gallon per Day Pumped
2.0 MGD = $ 6,000,000	$3.00/gpd
2.5 MGD = $ 7.100,000	$2.84/gpd
3.0 MGD = $ 8,100,000	$2.69/gpd
4.0 MGD = $10,100,000	$2.53/gpd
5.0 MGD = $12,100,000	$2.42/gpd
10 MGD = $21,800,000	$2.18/gpd
15 MGD = $27.400.000	$1.83/gpd
20 MGD = $33,600,000	$1.68/gpd
25 MGD = $38,900,000	$1.56/gpd
30 MGD = $42,900,000	$1.43/gpd
35 MGD = $46,800,000	$1.34/gpd
40 MGD = $50,800,000	$1.27/gpd
45 MGD = $54,900,000	$1.22/gpd

5. Sediment Removal Costs

The sediment removal cost presented in this report is based on $500 per ton, the unit cost used in the Jones Falls Alternatives Analysis Report.